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VOL. 22 DECEMBER, 1930 No. 12

THE PURIFICATION OF HIGHLY TURBID WATERS[1]

BY JOHN D. FLEMING[2]

The designation of waters as clear, turbid, or highly turbid, is but arbitrary and indefinite, just as are the terms soft, hard and very hard. The term highly turbid, however, quite generally brings to mind a water like that of the Mississippi River, the Missouri River or some of its tributaries, with a turbidity of at least 1000 p.p.m.

These waters are generally of low color, hard, highly polluted, and subject to very sudden as well as great variations in turbidity and suspended solids. The coefficient of fineness of the suspended solids is seldom less than 0.7, and a considerable amount of fine sand is always present.

It is the range and sharpness of the variations as well as the quantity of mud and sand carried by such waters that give rise to the chief problems of their purification, viz., basin capacity, sludge storage and sludge disposal. The application of chemicals and the conditioning of the freshly treated water must also be given special consideration.

Table 1 classifies highly turbid waters on the basis of turbidity and coefficient of fineness. The successive steps in the rational treatment of each class is indicated, both for softening with clarification and for

[1] Presented before the St. Louis Convention, June 3, 1930.

[2] Chief Chemical Engineer, Howard Bend Station, Water Division, St. Louis, Mo.

TABLE 1

Suggested procedures for purification of highly turbid waters

(1)	(2)	(3)	(4)	(5)	(6)
COEFFICIENT OF FINENESS, LESS THAN 1.1, TURBIDITY LESS THAN 1,200	COEFFICIENT OF FINENESS, GREATER THAN 1.1, TURBIDITY LESS THAN 1,200	COEFFICIENT OF FINENESS, GREATER THAN 1.1, TURBIDITY GREATER THAN 1,200 AND LESS THAN 2,000	COEFFICIENT OF FINENESS, LESS THAN 1.1, TURBIDITY GREATER THAN 1,200 AND LESS THAN 2,000	COEFFICIENT OF FINENESS, GREATER THAN 0.7, TURBIDITY GREATER THAN 2,000	COEFFICIENT OF FINENESS, LESS THAN 0.7, TURBIDITY GREATER THAN 2,000
A. Softening and clarification					
1. Chemical Dosing 2. Conditioning 3. Sedimentation 10 to 12 hours in special basins (part of bottom hoppered) 4. Carbonation 5. Secondary Coagulation (Prechlorination) 6. Sedimentation 4 to 6 hours in plain basins 7. Filtration 8. Chlorination	1. Grit Chamber 2. Chemical Dosing 3. Conditioning 4. Sedimentation 8 to 10 hours in special basins (part of bottom hoppered) 5. Carbonation 6. Secondary Coagulation (Prechlorination) 7. Sedimentation 4 to 6 hours in plain basins 8. Filtration 9. Chlorination	1. Grit Chamber 2. Natural Sedimentation 3 to 5 hours in special hopper bottom basins 3. Chemical Dosing 4. Conditioning 5. Sedimentation 8 to 10 hours in special basins (part of bottom hoppered) 6. Carbonation	Same as for Class 3 with omission of the Grit Chamber	1. Natural Sedimentation 3 to 5 hours in special hopper bottom basins or mechanically cleaned 2. Chemical dosing 3. Conditioning 4. Sedimentation 10 to 12 hours in special basin (part of bottom hoppered) 5. Carbonation	1. Chemical Dosing 2. Conditioning 3. Sedimentation 10 to 12 hours in special hopper bottom basins (part of bottom hoppered) 4. Carbonation 5. Secondary Coagulation 6. Sedimentation 4 to 6 hours in plain basins

	7. Secondary Coagulation (Prechlorination)			6. Secondary Coagulation (Prechlorination)	7. Tertiary Coagulation (Prechlorination)
	8. Sedimentation 4 to 6 hours in plain basins			7. Sedimentation 4 to 6 hours in plain basins	8. Sedimentation 4 to 6 hours in plain basins
	9. Filtration			8. Filtration	9. Filtration
	10. Chlorination			9. Chlorination	10. Chlorination
B. Clarification only					
The treatment in each case is the same except that carbonation is not included					

clarification only. These indications are, of course, merely the author's suggestions and are offered for the general cases and not specific ones.

GRIT CHAMBER

In most cases, where the coefficient of fineness of the suspended solids is above 1.1, considerable sand is present which makes the

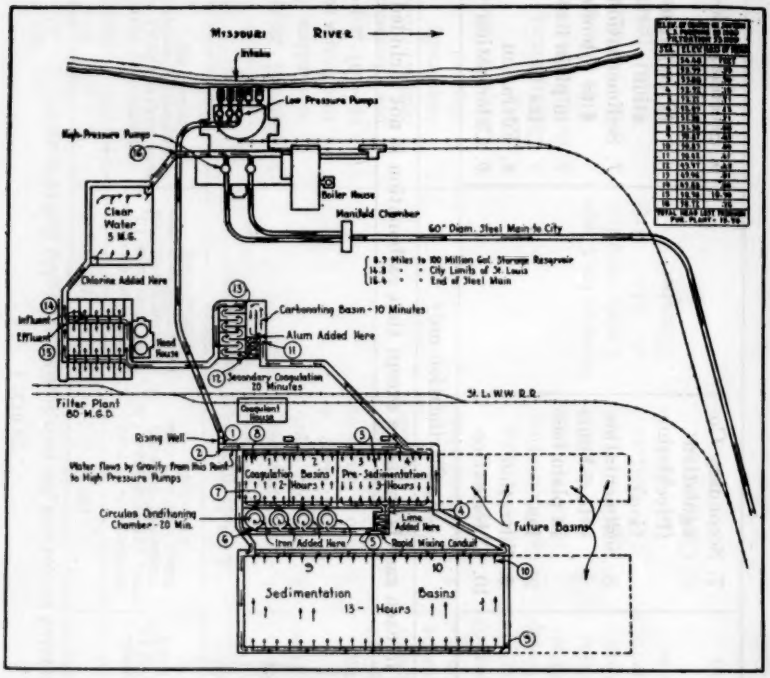


FIG. 1. DIAGRAM SHOWING OPERATION OF HOWARD BEND PURIFICATION AND PUMPING PLANT

cleaning of plain and even hoppered bottom basins difficult and costly. The grit chamber, always comparatively small is designed for easy and economical cleaning. A grit chamber is usually unnecessary if the settling basins are equipped with mechanical cleaning devices.

PRELIMINARY SEDIMENTATION

If the coefficient of fineness is greater than 0.7 a water will, in most practical cases, settle without chemical aid to a turbidity of 500

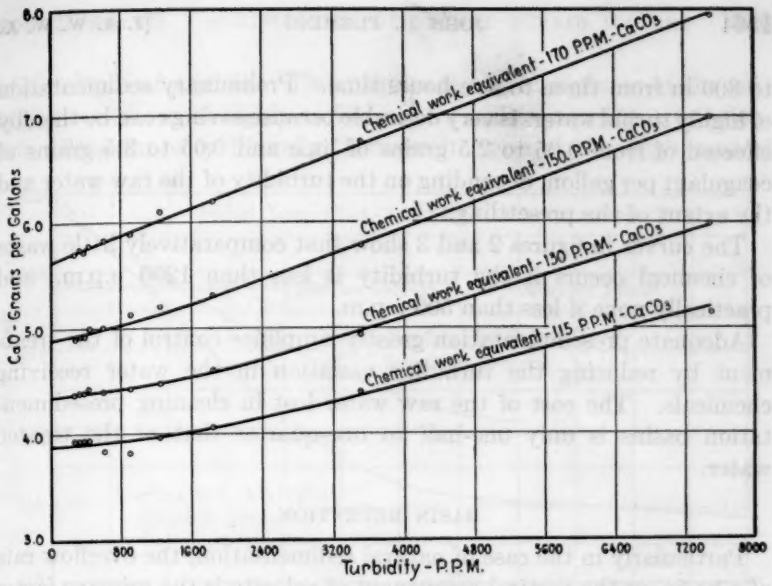


FIG. 2. CURVES SHOWING EFFECT OF TURBIDITY ON LIME DOSAGES REQUIRED FOR FOUR GIVEN AMOUNTS OF CHEMICAL WORK

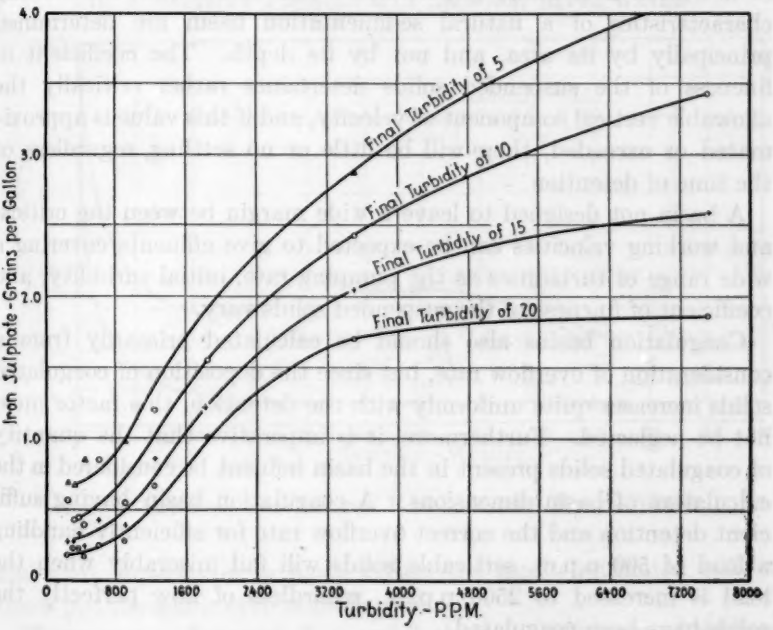


FIG. 3. CURVES SHOWING EFFECT OF TURBIDITY ON IRON SULPHATE DOSAGES REQUIRED TO YIELD WATERS OF GIVEN TURBIDITIES AFTER 18 HOURS SETTLING

to 800 in from three to five hours time. Preliminary sedimentation of highly turbid waters is very desirable because savings can be thereby effected of from 0.05 to 2.5 grains of lime and 0.05 to 3.5 grains of coagulant per gallon, depending on the turbidity of the raw water and the extent of the presettling.

The curves in figures 2 and 3 show that comparatively little waste of chemical occurs if the turbidity is less than 1200 p.p.m., and practically none if less than 500 p.p.m.

Adequate presedimentation greatly simplifies control of the treatment by reducing the turbidity variation in the water receiving chemicals. The cost of the raw water lost in cleaning presedimentation basins is only one-half to one-quarter that of the treated water.

BASIN DETENTION

Particularly in the case of natural sedimentation, the overflow rate of a basin, or the vertical component of velocity is the primary factor in determining the degree of settling, and the duration of the settling, or detention is of minor importance. This means that the settling characteristics of a natural sedimentation basin are determined principally by its area, and not by its depth. The coefficient of fineness of the suspended solids determines rather critically the allowable vertical component of velocity, and if this value is approximated or exceeded, there will be little or no settling, regardless of the time of detention.

A basin not designed to leave a wide margin between the critical and working velocities can be expected to give effluents covering a wide range of turbidities as the pumping rate, initial turbidity, and coefficient of fineness of the suspended solids vary.

Coagulation basins also should be calculated primarily from a consideration of overflow rate, but since the deposition of coagulated solids increases quite uniformly with the detention, this factor must not be neglected. Furthermore, it is imperative that the quantity of coagulated solids present in the basin influent be considered in the calculation of basin dimensions. A coagulation basin having sufficient detention and the correct overflow rate for efficiently handling a load of 500 p.p.m. settleable solids will fail miserably when the load is increased to 2500 p.p.m., regardless of how perfectly the solids have been coagulated.

Consideration of the load will logically lead to the determination

of the depth of a basin. The magnitude and abruptness of the turbidity variations of highly turbid waters makes it prudent to calculate all basins very liberally, because in spite of thorough control

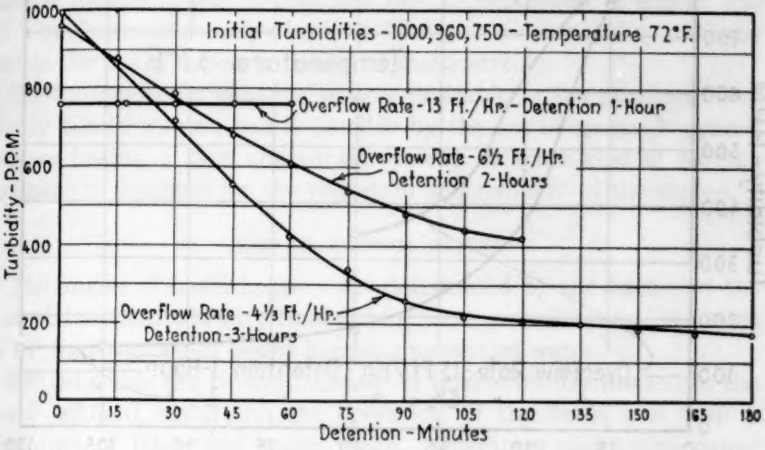

Initial Turbidities - 1000, 960, 750 - Temperature 72°F.
Overflow Rate - 13 Ft./Hr. - Detention 1-Hour
Overflow Rate - 6½ Ft./Hr.
Detention 2-Hours
Overflow Rate - 4⅓ Ft./Hr.
Detention - 3-Hours
Turbidity - P.P.M.
Detention - Minutes

FIG. 4. SETTLING TESTS ON RAW MISSOURI RIVER WATER

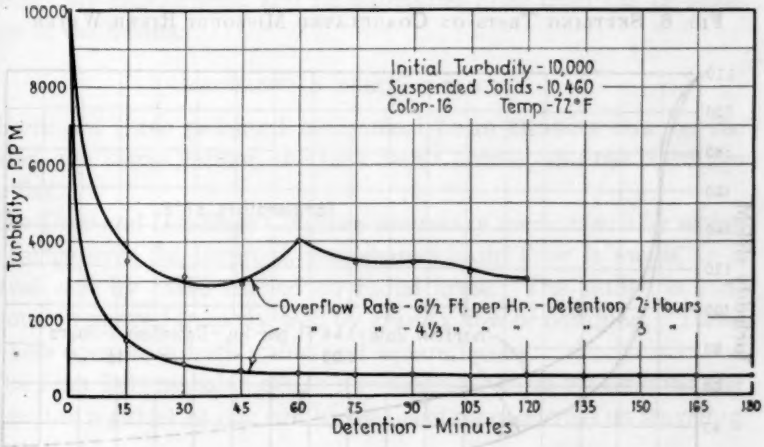

Initial Turbidity - 10,000
Suspended Solids - 10,460
Color - 16 Temp - 72° F
Overflow Rate - 6½ Ft. per Hr. - Detention 2 Hours
" " - 4⅓ " " " " 3 "
Turbidity - P.P.M
Detention - Minutes

FIG. 5. SETTLING CURVES FOR RAW MISSOURI RIVER WATER

and alert supervision, a basin layout too nicely estimated will at times put the whole system in distress.

The curves in figures 4, 5, 6 and 7 show the influence of various overflow rates and detentions on the settling of raw and coagulated water.

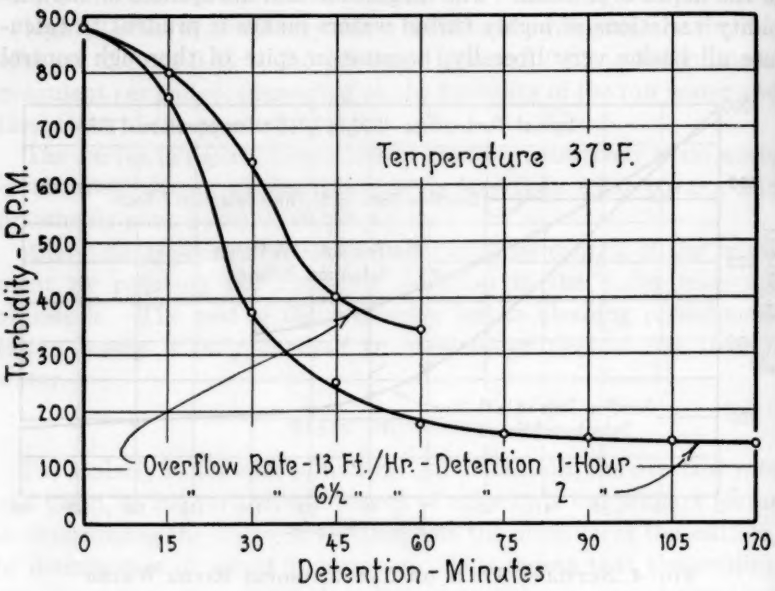


FIG. 6. SETTLING TESTS ON COAGULATED MISSOURI RIVER WATER

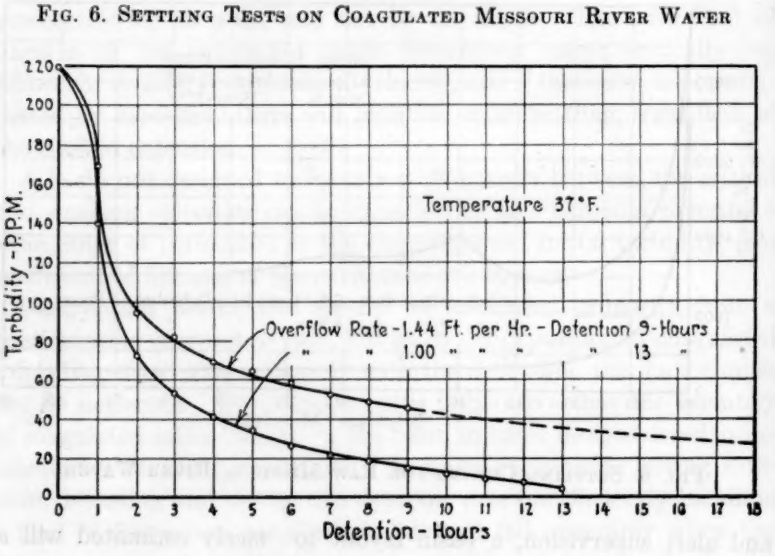


FIG. 7. SETTLING TESTS ON COAGULATED MISSOURI RIVER WATER

CONTINUOUS BASIN CLEANING

For each 1000 p.p.m. of suspended solids present, 1252 cubic feet of sludge containing 10 percent of solids will be deposited by each million gallons of water, and this rate of deposition of sludge will fill a twelve-hour capacity basin in 53 days. Such rapidity of filling makes the use of ordinary basins impracticable.

Continuous basin cleaning is very desirable in the purification of highly turbid waters, and is possible by the use of special hoppered bottom basins, a pipe underdrain system, or some kind of moving mechanism designed for the collection and removal of the sludge.

HOPPER BOTTOM BASINS

The angles of basin hoppers are determined by the nature of the material settled, and vary from 45° for some presedimentation basins to 10° for coagulation basins handling presettled water.

Settled solids can be continuously or intermittently flushed to the sewer without disturbing the operation of the basin and with a minimum of labor and water waste. Operation and maintenance costs of such basins are small, the efficiency high, no mechanical troubles are experienced, and no difficulties arise from the presence of ice in the basins.

MECHANICAL BASIN CLEANERS

There are three patented mechanical basin cleaners now on the market, the Dorr Clarifier, the Link Belt Clarifier, and the Hardinge Clarifier.

The Dorr and Hardinge Clarifiers are rotary mechanisms by which the sludge over the inverted cone shaped basin floor is swept to a central well by rakes carried on radial arms. The sludge is continuously removed from the well by gravity flow or pumping. These clarifiers operate in both round and square basins.

The Link Belt machine cleans a rectangular basin by scraping the sludge into a gutter at one end by means of rakes carried on traveling chains.

THE DORR CLARIFIERS AT HOWARD BEND STATION

Four 150 feet square basins equipped with Dorr Traction Clarifiers, are in use at the new Howard Bend Station. Two are operating on presedimentation basins and two on coagulation basins handling

lime softened water. The presedimentation clarifiers make one complete circuit in twenty-five and one-half minutes, and the coagulation clarifiers require forty-eight and one-half minutes. Even at these slow speeds the moving rakes and supports are slightly disturbing to the sludge, and the efficiency of the basins is somewhat impaired.

Solids having a coefficient of fineness of 8.81, have been handled by the clarifiers and the basins kept clean when receiving deposited solids at the rate of 750 tons per day for each presedimentation basin. The operation of the mechanism ordinarily requires no close attention, but yet, like all mechanisms, even though slow moving, it cannot be left unwatched, and it is part of the routine to inspect the clarifiers once each hour.

OPERATING DIFFICULTIES WITH DORR CLARIFIERS

If for any reason the clarifiers in the presedimentation basins must be stopped long enough to allow the rakes to become buried in sludge, it is impossible to start the mechanism and the basin must be drained and hand cleaned. This was experienced once when the mechanisms had been shut down for forty-eight hours.

The clarifiers at St. Louis as well as the ones operating in circular basins at Kansas City have shown themselves unable to cope with the rigor of winter. All efforts to keep the machines moving against the ever growing ice jams in the basins last winter were unsuccessful, and defeat was accepted when several members of two of the trusses succumbed to bending stresses for which they had not been designed. Had these members been adequate, it is still a matter of conjecture as to how many hours longer the tractor would have been equal to the steadily mounting load. And, were both the truss members and tractor designed to meet this extreme duty, there would seem to be an inherent fatuity in the operation of the clarifiers under such conditions.

The accumulation of 8 to 10 feet of ice in the basin by reason of the surface ice being constantly broken, not only reduces the detention of the basin, but the ice being in constant motion virtually converts the settling basin into a mixing tank, and the usefulness of the settling basin becomes negligible. After the thaw in February, all the Dorr basins had to be drained and flushed before the clarifiers could be started.

SLUDGE REMOVAL FROM DORR BASINS

Gravity discharge of raw water sludge through a single line is highly unsatisfactory and troublesome whether controlled by gate valve, orifice or swiveled pipe. Remembering that the sludge discharge line must be of sufficient size to handle the flow at times of maximum basin duty, and having reasonable water economy in mind, it can be safely stated that no single line will handle sludge from a presedimentation basin, either by gravity flow or pumping, without occasionally becoming blocked by the segregation of sand and heavy mud. The velocity of flow must always be above the point at which segregation will occur, and the only practical way this can be accomplished is by having from 2 to 8 small pipes, the combined capacity of which shall be equal to the maximum flow. Only as many lines should be used as are required to hold the desired sludge level in the basin, and such lines as are used are not to be throttled in any way, but have full pipe opening for discharge. Sludge from coagulation basins handling presettled water may be easily removed by gravity flow or pumping.

If sludge is to be recirculated for any purpose, or if the elevation or distance of the place of final disposal requires it, resort is had to pumping, by centrifugal, diaphragm, or ejector type pumps. Diaphragm pumps having adjustable stroke are installed at Howard Bend, but are no longer used. In their stead a 3-inch Schutte Koerting water jet eductor has been installed in each sludge line. These ejectors require no attention whatever and are much more satisfactory and economical to operate than the diaphragm pumps.

The sludge from these coagulation basins is returned to the raw water entering the presedimentation basins as it is found to accelerate sedimentation from 10 to 50 percent. The ejectors on the presedimentation basin sludge lines will be used as long as the mud is lagooned for filling in around the plant. A set of eight 2-inch pipes leading from the mud well of the basin and discharging by gravity directly to the sewer, will then be installed.

CHEMICAL DOSING

Highly turbid waters treated only for coagulation are dosed with alum or iron sulphate and a small amount of lime. Alum is usually favored over iron, but it is quite possible in many cases that iron would be preferred if the subsequent mixing were done to the best

advantage. The coagulation of waters of high turbidity is best performed at a pH above 8.0, hence the use of a small amount of lime.

The softening of highly turbid waters in addition to their clarification presents some interesting aspects. These waters usually contain enough magnesium to give a good magnesium floc, if sufficient lime is added to give from 25 to 60 p.p.m. causticity. The magnesium floc formed is often sufficient for coagulation if the water be given the proper conditioning.

The advantages accompanying the use of excess lime in the softening treatment are, first, the increased softening made possible thereby when followed later with carbonation; secondly, the coagulation

TABLE 2

Lime wasted by various turbidities

TURBIDITY OF RAW WATER	LIME WASTED IN GRAINS PER GALLON			
	Chemical work equivalent			
	115 p.p.m.	130 p.p.m.	150 p.p.m.	170 p.p.m.
p.p.m.				
0	0.00	0.00	0.00	0.00
250	0.02	0.02	0.03	0.08
500	0.03	0.03	0.08	0.16
1,000	0.05	0.09	0.17	0.32
2,000	0.18	0.27	0.43	0.63
4,000	0.56	0.73	1.03	1.27
6,000	1.00	1.23	1.63	1.90
7,500	1.30	1.60	2.10	2.38

incident to the precipitation of the magnesium makes the use of iron or alum usually unnecessary and the non-carbonate hardness of the water is not thereby increased; third, a high degree of sterilization is given the water before filtration if the detention period is twelve hours or more; and fourth, the cost of the whole treatment including carbonation is frequently less than that of the combined lime and iron or alum treatment without carbonation. The cost figured on the basis of the softening done is considerably less. Carbonate hardness can be reduced by the excess lime and after carbonation method to 20 p.p.m., and possibly lower.

Owing to the fact that the lime wasted in softening a highly turbid water depends (see table 2) not only on the turbidity of the water,

but also increases greatly with the lime concentration present in the water being treated, overdosing of a part of the water is contraindicated. Every plant trial of this procedure made on the turbid waters of the Missouri and Mississippi at St. Louis has given results inferior to those of straight dosing.

In the excess lime treatment of turbid waters, it is desirable to completely dissolve the milk of lime in clear water before applying as the coagulation effected thereby is decidedly improved. The complete solution of lime, at a charge of one grain per gallon, requires the use of about 1.7 percent of the water being treated. The saturating tank can be located on the grounds near the mixing chambers and the large amount of water required can usually be pumped from an adjacent sedimentation basin for a cost of about 0.4 cent for each million gallons of water being treated with 1 grain of lime per gallon.

It appeared from experimental work at Howard Bend that the usefulness of presedimentation basins might be enhanced by splitting the lime dosage so as to give the raw water a small charge. This is not inconsistent with the lime saving argument for presedimentation, for if not over one-half the bicarbonate alkalinity of a highly turbid water is neutralized with lime, the waste of chemical is inappreciable and apparently no more than that incurred in dosing a presettled water of turbidity 500.

Complete softening of highly turbid waters with lime and soda ash is rarely ever done because the noncarbonate hardness of these waters does not justify it.

Waters of high color coupled with high turbidity sometime defy all efforts at softening or clarification by lime alone or lime and iron, and for these, clarification alone, with alum and lime must be accepted, if the physical features of the plant do not permit of softening treatment after clarification has been effected.

Except in the unusual case just mentioned, iron sulphate has proved more economical than alum for the primary coagulation of Missouri and Mississippi River waters.

It has been the experience at the Chain of Rocks Station, treating Mississippi water, that the optimum mixing period for lime softening is much too long for coagulation with iron or alum and consequently at that station the coagulant is added last and allowed but comparatively little agitation.

Some remarkable results have been obtained in the laboratory with both ferric chloride and sodium aluminate in conjunction with

the excess lime treatment, and plant tests with these coagulants will be made at Howard Bend some time this year.

It has invariably proved more satisfactory and economical at the St. Louis plants to reserve a part of the coagulant for secondary application after the original high turbidity has been reduced to 25 or less by softening and primary coagulation. Even tertiary coagulation is expedient under some unusual conditions.

MIXING AND CONDITIONING

Mixing is a term which should be applied only to the process by which homogeneity of the water and applied chemical is established. Conditioning is the treatment by which flocculation of this homogeneous mixture is promoted, and consists of the gentle agitation of

TABLE 3

Iron savings by presettling to turbidity 500 p.p.m.

TURBIDITY OF RAW WATER	IRON SULPHATE SAVED IN GRAINS PER GALLON			
	Final turbidity after 18 hours			
	20 p.p.m.	15 p.p.m.	10 p.p.m.	5 p.p.m.
p.p.m.				
750	0.05	0.09	0.12	0.16
1,000	0.16	0.22	0.31	0.40
2,000	0.85	1.00	1.20	1.30
4,000	1.53	1.85	2.11	2.40
6,000	1.62	2.14	2.65	3.08
7,500	1.62	2.28	2.97	3.50

the mass for ten to forty minutes in such a manner as will give the suspended matter in the water a motion relative to the general motion of the water.

Flexibility of the conditioning system is essential both in respect to the duration and violence or velocity of the agitation given, if optimum results are to be regularly expected. It is particularly true of turbid waters that the two elements of conditioning, duration and velocity, must be rather nicely adjusted to suit the various combinations of chemical and physical properties of both the water treated and the applied chemicals. Turbidity, color, hardness, temperature, the chemicals applied and their amounts are factors affecting the results achieved.

Multiple or series tanks equipped with variable speed mechanical

agitators of the impeller or paddle types constitute the most approved form of conditioning device. No known type of fixed or constructed chamber or conduit can even theoretically meet all the requirements for ideal conditioning of turbid water undergoing softening or coagulation.

The multiple round conditioning tanks at St. Louis and Kansas City are known as tangential mixers and are in the class just referred to as fixed or constructed chambers. The water enters the tank tangentially through a side nozzle near the top and leaves at the center of the bottom. The velocity energy of the entering water is imparted to the mass of water in the tank, causing it to whirl and produce a general agitation. Any number of the four tanks at Howard Bend Station may be put into service and the duration of conditioning thus varied, but any change in the duration of the conditioning period is necessarily accompanied by a velocity change of the reverse order. The energy consumed in the agitation reveals itself and is measured by the loss of head through the tank. It is equal to or greater than the energy usually required for propelling mechanical agitators.

CARBONATION

Carbonation is an essential element in the satisfactory lime softening of any water, and it is a necessary counterpart to the excess lime treatment because the causticity of the water so treated must be neutralized in some manner before passing to the consumer. Carbon dioxide, specially prepared or taken as a waste product, is the cheapest acidifying agent available and singularly enough is the one most desirable. The proper use of carbon dioxide neutralizes the water's causticity after the purpose of such causticity has been served, and in so doing precipitates the excess lime as calcium carbonate. The small amount of carobnate hardness still remaining is stable and not apt to deposit in meters or hot water pipes.

Howard Bend Station is the largest plant softening a highly turbid water with excess lime followed by carbonation. Flue gas from the combustion of Illinois coal in a modern steam generating plant is scrubbed for removal of sulphur, compressed by a Nash rotary water sealed pump, and applied to the water through a perforated pipe distributor. Tests made on samples of the spent gas escaping from the water show only about 0.2 percent CO_2 by volume. No contamination of the water with phenol compounds in the gas

has been experienced, and in general the use of waste flue gas has been entirely satisfactory.

Carbonation, with its attendant precipitation of calcium carbonate, may be given just ahead of filtration without giving rise to any serious problems of sand incrustation, but this practice increases the load on the filters. The logical point of application, especially in a plant handling a highly turbid water, is from four to six hours before filtration. A small charge of alum added at the same time will facilitate the settling of the precipitated calcium carbonate.

PRECHLORINATION

The chlorination of a highly turbid water is a wasteful procedure and very apt to produce tastes which would not result from the application of the same charge to the settled or filtered water. Prechlorination of the settled water from two to twelve hours ahead of filtration is often advisable to reduce the bacterial load on the filters and to maintain the filter beds in their best condition. The practice is not warranted, however, in a plant using excess lime softening and having basin capacity sufficient to maintain a caustic condition of the water for twelve hours or longer. A high degree of bacterial and algal sterilization is thereby effected, and chlorination should be reserved for the filter effluent.

TREATMENT AT HOWARD BEND

Referring to the flow diagram, figure 7, the present treatment of the Missouri River water at Howard Bend is seen to be, (1) two hours presedimentation in two Dorr basins aided by the return of all the sludge ejected from the coagulation basins; (2), dosing with milk of lime (and iron sulphate or alum solution at times); (3), conditioning for twenty to thirty minutes in 4 tangential flow circular tanks; (4), sedimentation for two hours in 2 Dorr basins; (5), the addition of alum as secondary coagulant; (6), thirteen hours sedimentation in 2 plain basins operating in parallel; (7), ten minutes carbonation: (8), another dosing with alum; (9), conditioning for twenty minutes in a baffled basin; (10) filtration and (11), chlorination.

The average pumping at this station for the past six months has been over 50 million gallons per twenty-four hours.

After a year of operation the plant is just emerging from the chaos and metamorphoses of the tuning up period. With the operating force now fairly well organized and acquainted with the purpose and

behavior of the many treatment units, the future operation of the plant promises to be successful and interesting.

SUMMARY

1. The characteristics of highly turbid waters and the nature of their purification problems are given.

2. Turbid waters are classified on a basis of turbidity and coefficient of fineness and appropriate treatments are suggested. Use of the grit chamber and preliminary sedimentation are discussed.

3. Theoretical and practical considerations involved in determining basin dimensions and the logic of continuous basin cleaning are stated.

4. Hoppered bottom basins and mechanical cleaning devices are discussed and the operating experiences with Dorr Clarifiers and sludge pumps at Howard Bend Station are related.

5. The special problems of chemical dosing and conditioning of highly turbid waters are dealt with in some detail.

6. Carbonation and the proper use of prechlorination in treating turbid waters are explained.

7. The purification process at Howard Bend Station is briefly recited.

DISCUSSION

F. C. DUGAN:[3] Mr. Fleming has covered the subject of The Purification of Turbid Waters so well that there is little to add. He has stated that in the design of any successful water purification plant complete knowledge of the local conditions is necessary. That fact should be emphasized for even on the same river the character of raw water might be so different that the same type of purification could not be economically used on two supplies even within a few miles of each other.

It is impossible to say at what degree of turbidity generally natural sedimentation is not economical. That would depend on local conditions. Assuming, for example, that 1200 turbidity appears to be the point below which natural sedimentation is not considered economical, we may find one water which has a turbidity of 1100 for periods of several weeks throughout the year, while another might rarely reach that turbidity. In all probability, the use of natural sedimentation in the first case might be warranted, due to the saving of chemicals, while it would not be in the second.

[3] Chief Engineer, State Board of Health, Louisville, Ky.

Primary sedimentation in either plain sedimentation basins or in those equipped with some type of mechanical clarifier is economical on waters similar to those from the Ohio River.

At all of the filtration plants in Kentucky, securing water from the Ohio River, we have algae troubles. Louisville with her preliminary sedimentation basins, has largely overcome these troubles by the use of artificial turbidity. The mud which has settled out in the basins during the periods of high turbidity is pumped out and mixed with the water of low turbidity, but high algae content, as it comes into the basin. The period of sedimentation in the basin followed by coagulation and settling reduces the amount of algae, thereby giving longer filter runs. I mention algae trouble for that difficulty should be taken into consideration in designing a plant using raw water which will have a high algae content for a considerable period of time.

Mr. Fleming mentions pumping the sludge from the coagulation basin and mixing it with the raw water coming into the plant. Two of the plants on the Ohio River tried the same scheme in trying to reduce the algae content. The results were unsuccessful and besides the bacterial count went out of sight, placing an additional load on the filters. I am curious to know whether any increase in the bacteria count and the B. coli index occurred at the Howard Bend plant.

I believe that the trend in the practice of mixing chemicals for coagulation is towards stirring devices, because they will permit greater flexibility of control. Furthermore the need of longer periods of mixing to secure good flocculation is being realized more today than it has been in the past.

That laboratory control is necessary has been long recognized by the leaders in the water purification field, but it has been most difficult to make the owner of the small plant realize its advantages. This is true for both municipally and privately owned plants.

One of the greatest problems is the securing and retaining of competent men for the operation of small filtration plants. In many instances, in municipally owned plants, the superintendent is discharged when the administration changes and often a thorough, competent man is replaced by one who does not know alum from chlorine. In the privately owned plant, in many cases, it has been impossible to make the owners realize that the saving in chemicals and in the wear and tear on the equipment, will generally pay the difference in salary between a high grade man and one of small capacity.

DEVELOPMENT OF A WATER DISTRIBUTION SYSTEM FOR GREATER DETROIT[1]

By F. H. Stephenson[2]

This paper describes briefly the conditions necessitating the development of a comprehensive plan to distribute water to greater Detroit from three widely separated pumping stations, the methods of determining the requirements and the steps taken to devise a system to fit in with the existing one.

The source of Detroit's water supply is the Detroit River. In 1873 the first parcel of land at what is known as Water Works Park was purchased and since 1878 practically all water delivered to consumers has been pumped from stations located at this point. Other adjacent lands have been purchased as required to accomodate additional facilities for the collection, purification and distribution of the water supply.

In 1924 the Department of Water Supply was supplying the City of Detroit, with an estimated population 1,222,500, and adjacent territory, with an estimated population 139,000, or a total of 1,371,500 consumers. The average daily consumption was 183,974,000 gallons. The maximum hourly consumption was at the rate of 282,720,000 gallons per 24 hours and the minimum hour at the rate of 88,560,000 gallons per 24 hours. These varying demands were met by direct pumping to the distribution system as there is no sufficiently high ground near Detroit where a storage reservoir might be located. The highest point in the city is about 90 feet above Water Works Park and some 11 miles away in an air line.

To meet the demand for water there were at Water Works Park and vicinity, an intake and intake tunnel, a low lift pumping station which raised water to a filtration plant of 320 m.g.d. nominal capacity, a filtered water reservoir, a steam generating plant and two pumping stations which delivered water to the distribution system. The pumping equipment consisted of three compound beam type units built in 1877, 1880 and 1886 respectively and ten triple ex-

[1] Presented before the St. Louis Convention, June 6, 1930.

[2] Engineer of Water System, Department of Water Supply, Detroit, Mich.

pansion units of later date. These pumps discharged water into six 42-inch and seven 48-inch cast iron or steel mains, which in turn delivered water to the lateral or service mains.

When Water Works Park was chosen as the location of the water plant it was at the eastern end of a city of 12.75 square miles in area. In 1924 the area of the city had expanded more toward the west than toward the east to 92.66 square miles and 11.18 square miles of territory outside city limits were receiving Detroit water. This expansion brought about the separation of the territory served into service zones and certain pumps and mains were allocated to supply these zones. Control points were established in each zone and an hourly record of pressure gage readings is received at the pumping station by telephone. From this information the station operator regulates his pumping equipment.

For a number of years it was evident that the rapid commercial and industrial development of Detroit with its attendant increase in population and area to be supplied, would soon create a demand for water which the existing plant would be unable to meet. Engineers were employed to study the problem and submit reports and estimates on supplies from Lake Huron and Lake St. Clair. In 1921, Mr. Geo. H. Fenkell, Superintendent and General Manager, was directed to prepare a report on additional supply. Mr. Fenkell secured the services of Mr. Louis E. Ayres of Ann Harbor as principal assistant to collect and tabulate the data and to assist in the preparation of his report. This report was very complete and recommended that the source of a new supply be the Detroit River near the present source. The reports on Lakes Huron and St. Clair supplies were reviewed and the estimates of cost revised to enable consideration of all projects on a common basis. Mr. Fenkell's report covered the anticipated requirements of a total population of 3,500,000 people occupying about 300 square miles of area and his studies indicated that these figures would be reached between 1950 and 1960.

In the preparation of this report much data were collected which were later used in the distribution studies. There were available the annual reports of the Board of Water Commissioners from 1853 to date containing information on population, pumpage, use of meters, pressures, etc. The Michigan Bell Telephone Company allowed the use of its "Commercial Survey of Detroit and Environs." U. S. Census reports and yearly school counts by the Board of Edu-

cation were available. Reports of consulting engineers on sewage disposal and rapid transit were studied and information obtained from railroads and other public utilities.

FORECASTS OF POPULATION

It was necessary to forecast the probable population, its location and density, type of residents and their homes, the probable number of business places and manufacturing plants and any other pertinent data which tend to influence the amount of water consumption. It was possible to determine the past and present population by city wards and other political divisions, homogeneous areas, that is, areas containing similarity of character in type of structures such as cottages, single residences, duplex houses, flats and apartments, business and large manufacturing plants. Similar data from other large cities were studied and compared with Detroit data.

To determine the amount of water required, the per capita consumption of past years, the results of a continuous pitometer survey since 1919, which divides the city into 155 districts, and actual meter records of water consumption by representative consumers were studied and the probable future consumption predicted. These predictions were developed to determine the final estimates of the amounts required and location of the distributing points of the new supply.

With the receipt of Mr. Fenkell's report, the Board of Water Commissioners had for consideration reports made by four engineers covering seven projects from three sources of supply. The services of another engineer familiar with Detroit's problems were secured and the five were directed to make a study of all projects and submit definite recommendations for an additional supply. This body of consulting engineers recommended that the new supply be obtained from the Detroit River and be purified and distributed from points in the western and northeastern parts of the city. Improvements to the existing pumping plant and a new 60-inch high service main were also recommended. Work in accordance with these recommendations is now in progress.

PLANNING OF DISTRIBUTION SYSTEM

With an additional supply assured and the water to be distributed from the existing and two new pumping stations, it became necessary to plan a distribution system to meet the new conditions. The

services of Mr. Ayres were again secured to develop the distribution plan.

The possibility of one open system fed from three widely separated points with no closed valves was studied, but due to relative elevations of existing and proposed pumping stations and the areas to be served the idea was given up. Those in charge of operation were strongly in favor of separate zones because of their experiences when breaks occur in large mains.

The following values were assumed as reasonable friction losses in the pipe system.

	Loss in main *pounds per mile*
Minimum hour	0.75
Minimum day	1.50
Average	2.25
Maximum day	3.75
Maximum hour	5.50

About 50 pounds per square inch were assumed as satisfactory average pressure for the central portion of the city and 25 pounds per square inch in the outlying districts at times of maximum consumption. With all the new supply and a considerable portion of the present supply to be pumped by motor driven centrifugal units, it was decided to control pressures by maintaining a uniform pressure at convenient points approximately in the center of each service district. It was found that, with the conditions as above mentioned, the variation of pressure at the pumps would be within the economical range of the units.

The distribution system was laid out into four service areas. The down town business district, because of age and condition of existing mains, will be served from Water Works Park at a lower pressure than the other areas. The other areas will be served from the three stations, each station working on two pressures and each area served by two stations. The probable limits of these areas was determined and from the studies previously mentioned the amount of water required was calculated and the sizes of distribution mains determined.

Because of the large volumes of water to be handled, large feeder mains were required. Physical conditions are such that the use of two or more smaller mains of equal capacity is not practical. In general, steel or iron plates are used as material for all mains greater than 36-inch in diameter and cast iron for the other mains.

Each service area is fed by two or more mains with cross connections at frequent intervals. The several services are connected at their boundaries, but kept separate by closed and sealed valves. In case of necessity bleeding from one service to another is possible.

The four service areas cover about 280 square miles. For service beyond these limits booster stations will be required and their approximate location and size of discharge mains has been determined. One such station is already in operation.

With the great variation in demand which occurs in the Detroit area, even a comparatively small storage has been found beneficial. There have been erected and put into service, elevated steel storage tanks of 1.5 million gallons capacity. Ordinarily these tanks are allowed to ride on the line when they exert a steadying influence on pumps and pressures. At times of high consumption the tanks are filled in the late evening hours and the stored water is held in reserve to be eased into the system in the late afternoon when the demand is at its peak. This method of operation has been very satisfactory.

A large part of greater Detroit was laid out in townships, sections and quarter sections, with main highways on the mile and half mile lines. In general it is the intention to bound each square mile with not less than a 12-inch pipe and in the half mile roads lay not less than an 8-inch pipe. Extra wide streets will have a pipe line on each side. In the intermediate streets 6- and 8-inch pipes are laid with more 8- than 6-inch. In the older sections of the city, advantage is taken of street resurfacing or widening to kill old mains and replace with larger ones, principally 8- or 12-inch. A recently completed hydrant flow survey covering over 21,000 hydrants, shows very few places where the available water supply is not sufficient to meet the demands that may be required to extinguish any fire that may occur. These weak spots will be strengthened as rapidly as possible.

With about 80 pressure gages scattered over the district which are read hourly, it has been possible to plat on a map the pressures at a given time and draw lines of pressure similar to contour lines, also lines of friction losses taking into account the differences of gage elevations.

A study of these is useful in locating districts that may need reinforcement. The department also calls on its pitometer bureau to measure flows in trunk mains and determine friction losses.

The developed distribution plan is called the "master plan," not necessarily to be rigidly followed as shown, but to serve as a general

guide particularly in the development of new areas. The principal changes are generally changes in route of large mains to avoid heavy traffic and existing sub-surface structures. The plan is periodically revised to keep it up to date. As soon as completed new mains are placed in service, receiving water from the present system and helping to carry the load till the new supply is available.

DEVELOPMENT OF THE WATER DISTRIBUTION SYSTEM FOR A GREATER ST. LOUIS, MO.[1]

By Thomas J. Skinker[2] and Meyer Serkes[3]

It is only through the scientific control and distribution of water that the people of this country, with its great industries, have been able to enjoy such an important position among the nations of the world. Without waterworks there could be no great centers of population and wealth, or even so great a number of medium sized cities and towns as there exists today. Neither would it be possible for gigantic industries to operate and produce as efficiently as they do today.

The early growth and development of the St. Louis waterworks has been typical of most municipal plants, in financial and political entanglements.

Our early history shows that about 1829, when the population of St. Louis was 5,852, two enterprising citizens, by name, J. C. Wilson and Abraham Fox, entered into an agreement with the city to build and conduct a water works. It was stipulated that the supply to the citizens should be "clarified water." The source of the supply was the "Muddy Mississippi River." A settling basin 80 by 40 by 12 feet was excavated near the pumping station at the foot of Dickson Street. The pumping plant consisted of two rotary pumps, that were originally built for fire engine use. However, it seems that the settling basin was never used as the water was pumped directly from the River. Later a reservoir 62 by 55 by 15 feet was built at Collins and Bates (now Dickson) Streets, with a capacity of 350,000 gallons. Water was pumped to this reservoir through a 6-inch cast iron main, and from there it flowed by gravity to the city distribution system, which consisted of 2-, 3- and 4-inch mains, together with 12 fire hydrants.

In 1835 the city purchased the waterworks and expanded and improved it to meet the rapidly increasing demands. In 1871 the

[1] Presented before the St. Louis Convention, June 6, 1930.

[2] Division Engineer-in-Charge, Water Department, St. Louis, Mo.

[3] Assistant Engineer, Water Department, St. Louis, Mo.

"New Waterworks" as it was termed was built at Bissell's Point, or No. 34 East Grand Ave., on the Mississippi River. A large capacity reservoir was built at Compton Hill, which is now the site of an 88 million gallons storage reservoir. This reservoir floats on the system that supplies the eastern and south-eastern parts of the City.

In 1892 the "Chain of Rocks Plant" was developed, 7.5 miles north of the Bissell's Point Plant, on the Mississippi River. Here in 1914 a modern filter plant of 160 million gallons capacity was built. From these filters the water flows by gravity through two large conduits of 200 million gallons daily capacity, along the river front to the storage basins at Baden and Bissell's Point, to the high service pumps.

The following are the total capacities of these pumps—

	m.g.d.
Chain of Rocks Low Lift Pumps	300
Chain of Rocks Filters	160
Baden Pumping Station	80
Bissell's Point Station	140

In 1924, to meet the rapidy increasing demands for water, the recently completed Howard Bend Plant was started. This plant is located on the Missouri River, 16 miles west of the present city. The site for this new plant was chosen because it lay in a direct path with the future growth of Greater St. Louis, and the topography of the land was such as to utilize a very high point for the location of the 105 million gallons reservoir about half way between the city and the plant. From here the supply enters the city by gravity through a 62-inch riveted steel conduit. This conduit connects with the present distribution system at very strategical points for economical distribution.

The present capacity of the Howard Bend Plant is as follows:

	m.g.d.
Low Lift Pumps	240
High Lift Pumps	120
Filters	80
Pressure Supply, conduit to reservoir	70
Gravity Supply, conduit to City	70

In order to provide economically and efficiency for the present and future deveopment of Greater St. Louis, a scientific study of the

distribution system was made by the Pitometer Company. In this study valuable and accurate information as to population and growth was obtained from individual studies made by the Rapid Transit Commissioner the Southwestern Bell Telephone Company of this city, and the United States Government Census.

In this study the past growth, the present trend of growth and the area available for expansion were taken into consideration. Great weight was given to the zoning ordinances adopted by this city. These affect the character and growth of certain districts, and are determining factors for the future trend of population and industry.

The outlook for the future growth of St. Louis is very promising. It has excellent transportation facilities, both rail and air; a nearby coal supply; an unlimited source of water supply; and plenty of available space in and near the city for the growth of its industries and population. From all indications St. Louis will probably grow steadily in the future, as it has done in the past, although in common with most large cities the rate of increase will be more gradual. The annexation of St. Louis County will, of course, change this rate of growth.

Most of the industries of this city are located along the Mississippi River and the Mill Creek Valley. There is considerable room for additional growth along these sites. In the northwestern part of the city, along the Harlem Creek drainage basin, a newer industrial area has been developed. This section is given over largely to automobile body and assembling plants, manufacture of drugs, wire rope and stoves. It is here that the General Electric Company plan to erect a very large plant, which will use from 5 to 10 million gallons of water per day. There is ample room for expansion in this section.

In the downtown district the light manufacturing industries, hats, shoes, clothing, etc. are located. This district is well occupied, and any expansion in these industries would have to take place to the north, south and west in old residential districts.

A greater part of the population growth will take place in the northwestern, southern and southwestern parts of the city where vacant areas are steadily being developed. In the central western part of the city, which contains very little vacant land, the population will increase, due to the replacement of smaller dwellings by large apartment houses. In the central downtown district it is probable that the population will decrease. In this section there is practically no vacant land and the growth of the business district will occur by

encroachment on the areas now occupied by residence. As the land in the city available for residential purposes grows less, many residents of St. Louis City will move into St. Louis County. For this reason St. Louis County should grow at a more rapid rate than the city itself.

PRESENT AND FUTURE CONSUMPTION

The results of these various intensive studies lead to the following prediction on future population, for which an adequate water supply must be provided.

Population estimate

YEAR	CITY OF ST. LOUIS	CITY AND COUNTY OF ST. LOUIS
1930	873,364	1,005,973
1940	968,703	1,147,887
1950	1,058,914	1,306,364
1955	1,101,455	

In the water waste survey of the entire city recently completed, the city was divided into seven sections and these sections were divided again into districts. The amounts of water entering these districts were measured. Where excessive consumption seemed to exist, house to house inspections were made of all plumbing fixtures and service connections. In the district measurements the average daily consumption and the maximum hourly consumptions were obtained. These were used as a basis for estimating the water requirements up to the year 1955. The per capita consumption for each district measured was figured. These ranged from 90 to 275 gallons per day. Those districts which showed excessive per capita consumption and could not be reduced materially by house to house inspections were recommended for complete metering. The City of St. Louis is only 8.6 percent metered, the average per capita consumption for the city being 137 gallons per day.

The per capita consumption of the city for the past and how it probably will increase in the future is shown in figure 1. A gradual increase from 141 gallons per capita per day for 1930, to 151 gallons in 1955 is indicated in this figure. With this information it is possible to compute readily the estimated average daily, the maximum daily and the maximum hourly consumptions for five year periods up to 1955. The maximum daily consumption was estimated at

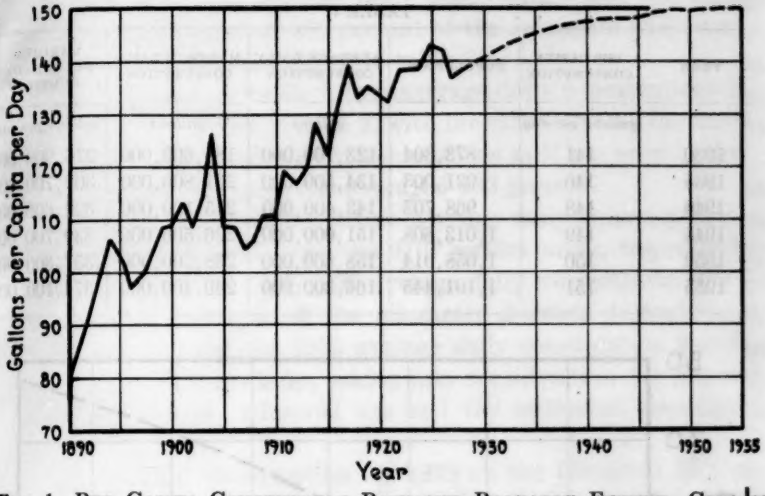


FIG. 1. PER CAPITA CONSUMPTION PAST AND PROBABLE FUTURE, CITY OF ST. LOUIS, MO.

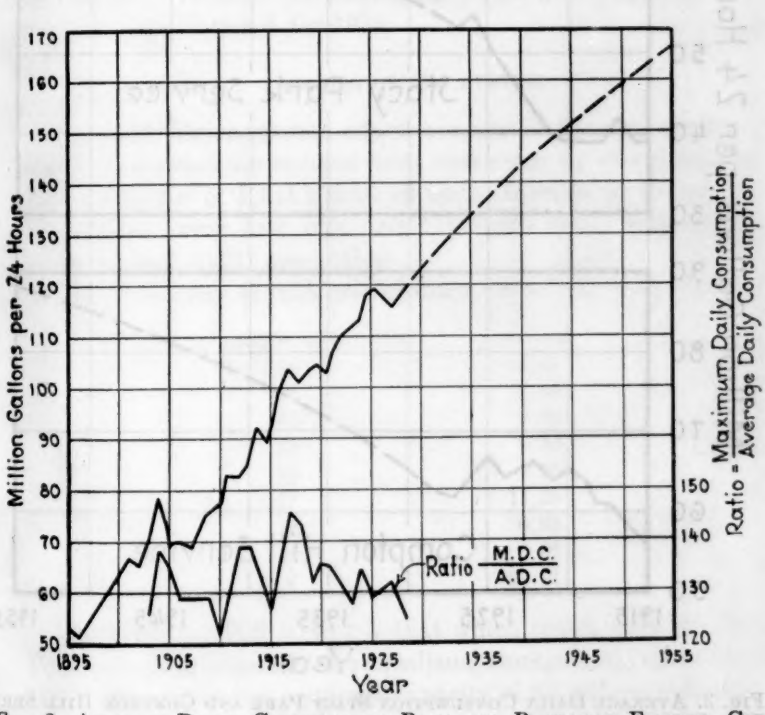


FIG. 2. AVERAGE DAILY CONSUMPTION PAST AND PROBABLE FUTURE, CITY OF ST. LOUIS, MO.

TABLE 1

YEAR	PER CAPITA CONSUMPTION	POPULATION	AVERAGE DAILY CONSUMPTION	MAXIMUM DAILY CONSUMPTION	MAXIMUM HOURLY CONSUMPTION
	gallons per day		*gallons*	*gallons*	*gallons per day*
1930	141	873,304	123,100,000	184,600,000	276,900,000
1935	146	921,003	134,500,000	201,800,000	302,700,000
1940	148	968,703	143,400,000	215,100,000	322,600,000
1945	149	1,013,808	151,000,000	226,500,000	339,700,000
1950	150	1,058,914	158,800,000	238,200,000	357,300,000
1955	151	1,101,445	166,300,000	249,400,000	374,100,000

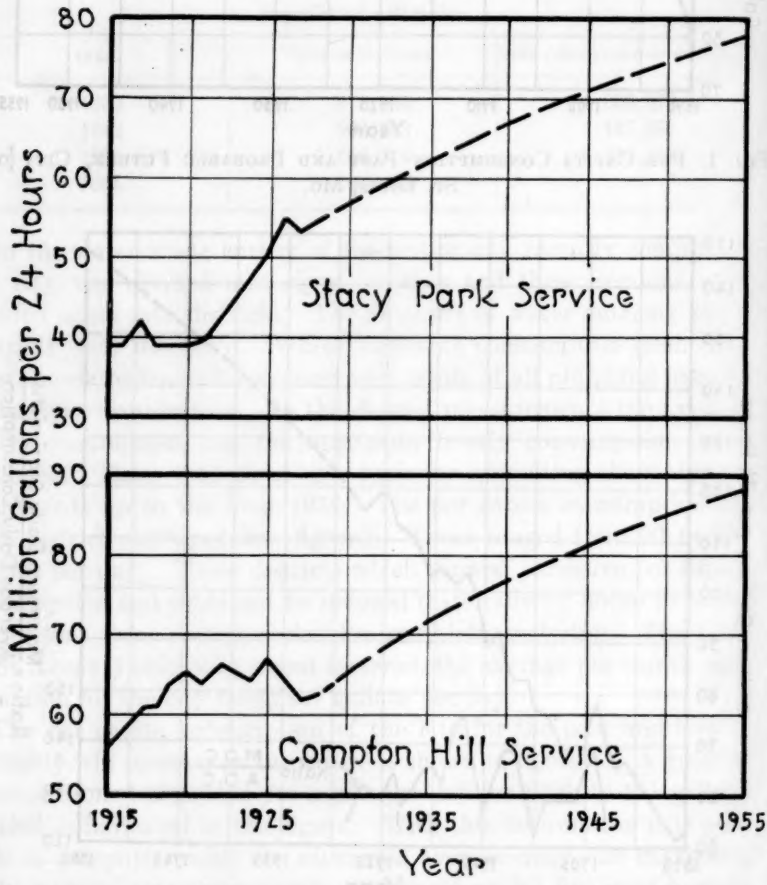


FIG. 3. AVERAGE DAILY CONSUMPTION STACY PARK AND COMPTON HILL SERVICES, PAST AND PROBABLE FUTURE, ST. LOUIS, MO.

150 percent of the average daily consumpion, and the maximum hourly consumption at 150 percent of the maximum daily consumption. The figures are shown in table 1.

The past and probable future average daily consumptions for the entire city are shown in table 2, with the ratio between the maximum daily and the average daily consumptions as it has been in the past. This ratio for the past has averaged 132 percent.

The past and probable future average daily consumptions upon the Stacy Park and Compton Hill service areas, respectively, are shown in figure 3. The 1930 average daily consumption was based on readings taken in all the pitometer districts during the water waste survey, and the 1955 average daily consumption was figured from these 1930 readings, taking into consideration the present and estimated future industrial use and the estimated population for 1955.

The average consumption for 1930 on the Compton Hill service amounts to 66 m.g.d. and for 1955 the estimate reaches 88 m.g.d. On the Stacy Park service the average consumption equaled 57 m.g.d. for 1930 and 77 m.g.d. for 1955.

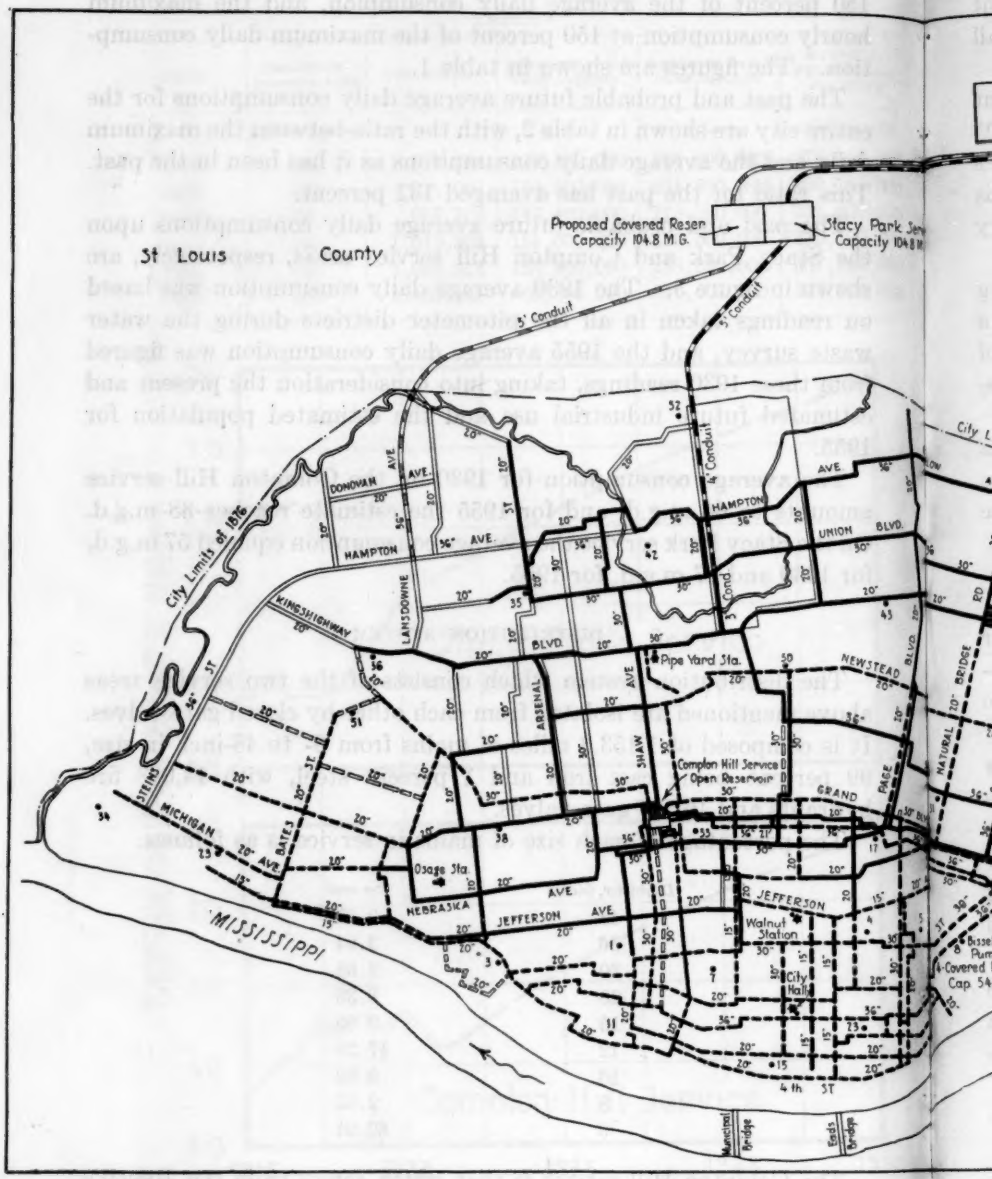
DISTRIBUTION SYSTEM

The distribution system which consists of the two service areas above mentioned are isolated from each other by closed gate valves. It is composed of 1,153.3 miles of mains from 6- to 48-inch in size, 99 percent being cast iron and 1 percent steel, with 14,083 fire hydrants and 16,547 gate valves.

The percentage of each size of mains in service is as follows:

Diameter, inches	*Per cent*
48	0.22
36	3.94
30	2.68
20	8.39
15	0.90
12	17.29
10	0.83
8	2.85
6	62.91

The Compton Hill service is that which comes from the Bissell's Point pumping plant and the equalizing storage basin upon Compton Hill. This basin has a storage capacity of 88 million gallons. The


St Louis County
Proposed Covered Reser
Capacity 104.8 M.G.
Stacy Park
Capacity 104.8
5' Conduit
4' Conduit
City Limits of 1876
DONOVAN AVE.
HAMPTON
56" AVE.
KINGSHIGHWAY
LANSDOWNE
BLVD.
ARSENAL
AVE
SHAW
Pipe Yard Sta.
Compton Hill Service &
Open Reservoir
NEWSTEAD
GRAND
PAGE
UNION BLVD.
NATURAL BRIDGE
STEINS
MICHIGAN
BATES
AVE.
Osage Sta.
NEBRASKA
JEFFERSON AVE.
JEFFERSON
Walnut Station
City Hall
MISSISSIPPI
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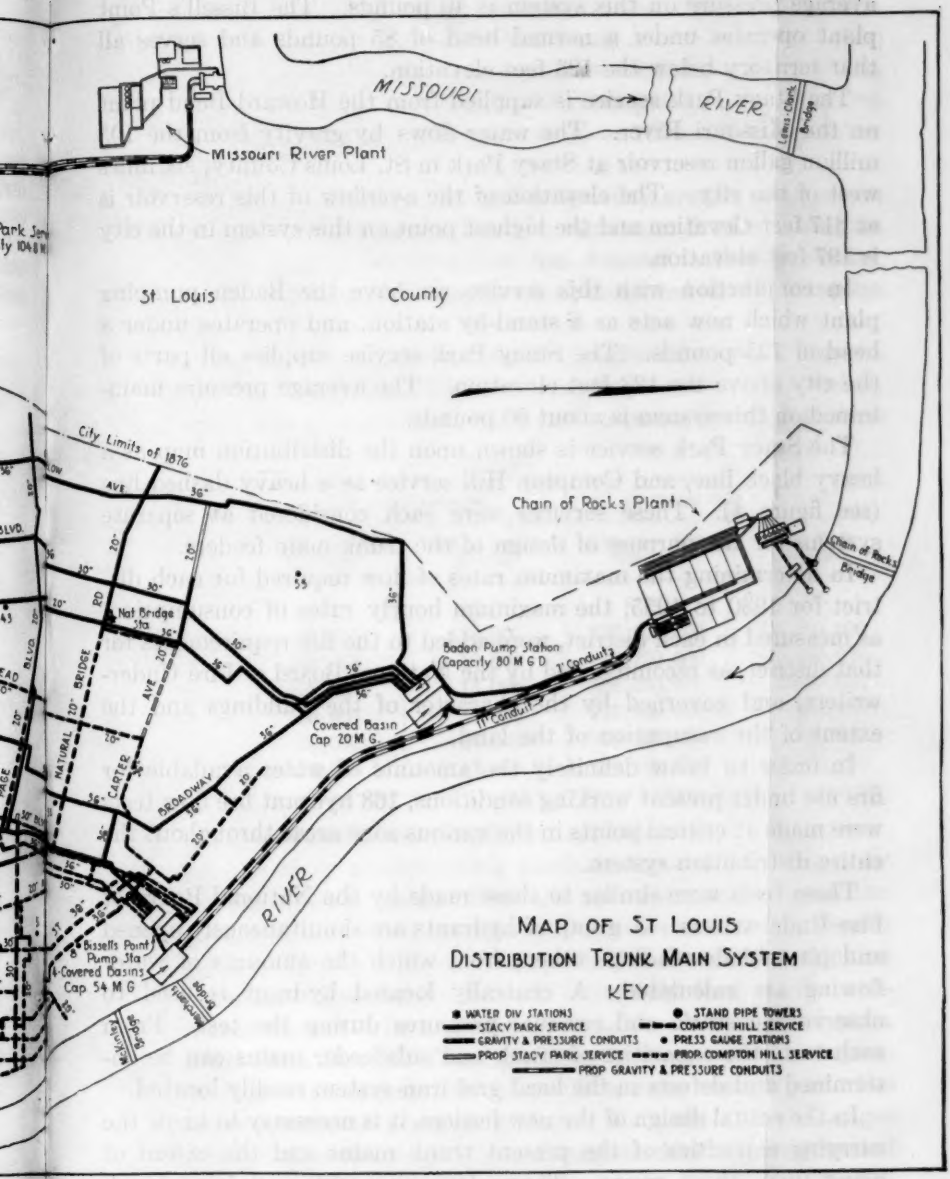
Municipal Bridge
Eads Bridge


MISSOURI
RIVER
Lewis Clark Bridge
Missouri River Plant
St Louis
County
City Limits of 1876
Chain of Rocks Plant
Chain of Rocks Bridge
Nat Bridge Sta
Baden Pump Station Capacity 80 M G D
7' Conduits
11' Conduit
Covered Basin Cap 20 M G
NATURAL BRIDGE AVE
CARTER
BROADWAY
RIVER
Bissells Point Pump Sta 4-Covered Basins Cap 54 M G
MAP OF ST LOUIS
DISTRIBUTION TRUNK MAIN SYSTEM
KEY
WATER DIV STATIONS
STAND PIPE TOWERS
STACY PARK SERVICE
COMPTON HILL SERVICE
GRAVITY & PRESSURE CONDUITS
PRESS GAUGE STATIONS
PROP STACY PARK SERVICE
PROP COMPTON HILL SERVICE
PROP GRAVITY & PRESSURE CONDUITS

average pressure on this system is 40 pounds. The Bissell's Point plant operates under a normal head of 85 pounds and serves all that territory below the 123 feet elevation.

The Stacy Park service is supplied from the Howard Bend plant on the Missouri River. The water flows by gravity from the 105 million gallon reservoir at Stacy Park in St. Louis County, six miles west of the city. The elevation of the overflow of this reservoir is at 317 feet elevation and the highest point on this system in the city is 197 feet elevation.

In conjunction with this service we have the Baden pumping plant which now acts as a stand-by station, and operates under a head of 125 pounds. The Stacy Park service supplies all parts of the city above the 123 feet elevation. The average pressure maintained on this system is about 60 pounds.

The Stacy Park service is shown upon the distribution map as a heavy black line, and Compton Hill service as a heavy dashed line (see figure 4). These services were each considered as separate systems for the purpose of design of the trunk main feeders.

In determining the maximum rates of flow required for each district for 1930 to 1955, the maximum hourly rates of consumption, as measured in each district, were added to the fire requirements for that district, as recommended by the National Board of Fire Underwriters, and governed by the character of the buildings and the extent of the occupation of the land.

In order to know definitely the amounts of water available for fire use under present working conditions, 168 hydrant fire flow tests were made at critical points in the various zone areas throughout the entire distribution system.

These tests were similar to those made by the National Board of Fire Underwriters. A group of hydrants are simultaneously opened and pitot blade readings taken, from which the amounts of water flowing are calculated. A centrally located hydrant is used to observe the static and residual pressures during the test. From such tests the capacities of trunk and sub-feeder mains can be determined and defects in the local grid iron system readily located.

In the actual design of the new feeders, it is necessary to know the carrying capacities of the present trunk mains and the extent of aging upon these mains. These data were obtained from trunk main surveys and friction loss tests made upon both systems during the years 1917 and 1928. The flows were obtained by use of the

pitometer and the head losses by special sensitive gauges which could be read as close as one-half foot head.

With these data at hand it was possible to calculate the amount of water required upon the Compton Hill and Stacy Park services for future designing in five year periods up to 1955. Table 2 shows these requirements.

The maximum rates of flow in gallons per day are the sums of the maximum hourly consumption and the maximum fire requirements on this service. The 20,000 g.p.m. fire demand is that necessary for the high value congested district which is located on the Compton Hill service.

The detailed study of the trunk main capacities from the Compton Hill Reservoir and the Bissell's Point pumping station, showed that, under conditions of combined maximum consumption and fire requirements, an insufficient supply is available from the Reservoir. This situation is at present being partially corrected by the utilization of two 20-inch mains which are now dead ended upon the Stacy Park service. These mains will be connected to two 20-inch mains leading directly from the Compton Hill Reservoir, which at present are also deadended, but will ultimately lead to the congested value district. In addition to this, the 36-inch steel main which leads from Bissell's Point pumping station to the Compton Hill Reservoir without any take-offs will be cross connected to the distribution system at strategic points. With the above changes and the recent pipe cleaning of several large feeders to the congested value district it will be possible to postpone the laying of a 36-inch main which was proposed in Geyer Avenue from the Compton Hill Reservoir to Menard Street.

TABLE 2

YEAR	FIRE REQUIREMENTS		MAXIMUM RATES OF FLOW	
	Compton Hill Service	Stacy Park Service	Compton Hill Service	Stacy Park Service
	g.p.m.	*g.p.m.*	*g.p.d.*	*g.p.d.*
1930	20,000	13,000	177,800,000	146,700,000
1935	20,000	13,000	189,700,000	156,600,000
1940	20,000	13,000	201,600,000	168,500,000
1945	20,000	13,000	210,800,000	176,600,000
1950	20,000	13,000	220,100,000	184,700,000
1955	20,000	13,000	229,100,000	192,800,000

The area between Vandeventer, Newstead, Page and Forest Park Boulevard, which is now on Compton Hill service, will be changed to Stacy Park service within the next 25 years, when additional supplies will be available from the Howard Bend Plant.

When this change in pressure zones take place, it will be necessary to change the 20-inch main in Page Boulevard, from Newstead to Grand Avenue, from the Compton Hill service to the Stacy Park service; and the 20-inch Stacy Park service main in Evans Avenue, between these points to Compton Hill service, connecting to the 20-inch main at Newstead and the 36-inch in Grand Avenue.

The Stacy Park service from the new plant enters the city in the central western part through a 62-inch steel conduit on the south side of Forest Park. Here it connects to a 36-inch main leading North and South at Hampton Avenue. At this point the 62-inch steel conduit reduces to a 48-inch steel main and extends eastward to Macklind Avenue where it connects to a 30-inch main leading North and South. From this point the 48-inch steel main reduces to a 36-inch cast iron main, which connects to a 30-inch main leading Northeast and South on Kingshighway Boulevard.

This new supply takes in the load of the Baden Pumps, and distributes it to the southwest and southeastern parts of the city, which are being rapidly developed.

This supply from the new plant, in conjunction with the total output from the Baden Pumping Station, has been estimated to take care of the needs of the Stacy Park areas to approximately 1940. After this time an additional 62-inch main will be needed to adequately take care of the fire protection and maximum hourly demands on this service.

This additional supply main will enter the city at about Eichelberger or Loughborough Avenue, which is that part of the city open for expansion, and will be adequately connected and looped with the existing distribution system.

One of the major extensions, at present, for this part of the city, is the continuation of a 36-inch trunk main from Manchester and Sulphur Avenue, over Hampton Avenue, to the south and thence eastward in Stein Street. This will connect with the Compton Hill Service which will ultimately be turned into Stacy Park Service in the extreme southeastern part of the city. From this main additional trunk mains will extend west and east, in conjunction with extensions to the east from the 30-inch Macklind Avenue main.

Ample provision is being made at all critical city limit points, for large capacity mains to inter-connect with the existing St. Louis County Water System, in the event of annexation. A major supply main already has been provided for at the Stacy Park Reservoir for a direct connection into this system.

In the detailed layout of this future system, the basic principles of distribution design were adhered to. Primary or trunk main feeders were spaced approximately 3,000 feet apart and are to be gated every quarter of a mile. The secondary feeders, which form the network of pipes of intermediate sizes and reinforce the distributor grid within the boundaries of the primary feeder systems and aid in the concentration of the required fire flows at any point, were likewise systematically spaced. In the primary and secondary feeders it was very important that they be looped and cross-connected wherever possible and gated at such intersections on all four sides. Such an arrangement greatly improves the capacity and efficiency of the system and is a guarantee against interruption of the supply during breaks or for temporary shut downs that may be required for large construction projects as bridges, sewers, and streets that may develop in the progress of the city.

These mains which are the keystones of the distribution system should be well guarded and cautiously installed at critical points where breaks or settlement may occur. The minor distributors or gridiron system which lie within the primary and secondary feeders, should consist of 6-, 8-, 10- and 12-inch mains.

INVESTIGATION OF GRIDIRON SYSTEM

With ample provision for capacity supplies made in the design of trunk mains an intensive investigation of the distribution gridiron system was next started. In this study the city was divided into small sections bounded by trunk mains. Each of the areas was examined critically with special consideration given to the present and future type of occupation, present water pressures, and the results of hydrant flow tests made therein. Where inadequate supply was indicated mains of proper size was recommended to bring the area to the proper required efficiency both for maximum consumption and for maximum fire requirements. In a good many cases 6-inch mains were found much longer than the 600 foot limit set by the Underwriters. It would have been too drastic to recommend that all these mains be replaced by larger ones. However.

each case was considered separately in the light of its present and future development, and where larger mains were actually needed such recommendations were made. Reinforcement of the existing grid in many cases is to be accomplished by laying mains in cross streets. In many cases by-pass connections from one main to another are to be made and a sufficient number of valves installed at each intersection. These are to be installed in such a manner that, in no single case of an accident, breakage, or repair to the system exclusive of arteries of the primary mains, will it be necessary to shut off from service a length of main greater than 500 feet in the high value districts, or greater than 800 feet in the other sections; and will not result in the shutting down of a primary main.

In the past few years the two-way system of main installation has been adapted for wide boulevards and streets. Such installations have proven beneficial in the reduction of street maintenance costs and a great public convenience upon heavily trafficed streets. They provide additional capacity for better fire protection, better service to consumers, reduction in electrolytic action, and shorter house service connections which tend towards elimination of underground leakage.

The proper location and installation of fire hydrants were given careful study. Although a system may have adequate carrying capacity to furnish the required fire flows, it frequently cannot be utilized to advantage on account of the lack of hydrants in their proper location. No better spacing could be adopted than that recommended in the Code of the National Board of Fire Underwriters, which gives the average area in square feet that each hydrant should control for engine streams or direct hydrant streams, and for the different character of districts. Symmetry and system should be observed in the actual location of hydrants. They should be placed at intersections in order to receive the full support of cross feeder mains, and also facilitate quick location by fire fighting forces. They should be arranged symmetrically along mains, and staggered to reduce the necessity of carrying hose leads across the streets. The connection to the street main should be not less than 6 inches in diameter. Those hydrants located in the high value, commercial, and industrial districts should be provided with a valve near the main in case of serious damage to the hydrant. A systematic and accurate record of all mains, valves, fire hydrants, special fittings, and appur-

tenances, which form a part of the distribution system are kept in the distribution office and when not in use are stored in a fire-proof vault.

A very important feature in the observation of the performance of the St. Louis distribution system has been the installation of 35 pressure recording guages in fire engine houses at critical points on the system. From these pressure charts the actual strength and capacity of the system has been observed during maximum periods of consumption or fire demands. They also act as an aid in locating peculiar conditions that may occur in pump operations or local disturbances.

This major program for the efficient and scientific development of the water distribution system for greater St. Louis, has been grouped systematically and in logical order of its importance and cost, into five year periods of construction up to 1955.

The benefits of such planning are manifold. Great savings in fire insurance premiums may be had by the community. Economic waste resulting from loss in business and employment, and in property damage can be markedly reduced. Higher standards of health can be maintained, better service to the consumers rendered and economies effected in street maintenance work. In fact, such planning is a positive prevention of water famines or shortages.

DISCUSSION

V. BERNARD SIEMS:[1] The paper by Mr. Stephenson is very clear and to the point. It illustrates a logical method of studying the requirements of distribution systems. In the third paragraph he makes mention of the average daily consumption. In calculating the future demand, is any allowance made for conservation work, or is the per capita consumption projected according to the past trend? One point seems rather striking, and that is the liberality of design as regards capacity of the plant. This is one of the most difficult features to justify economically, and yet in most cases the practice of designing for insufficient capacity cannot be justified economically. Some amplification by the author in discussion of this point would be of interest. He speaks of large steel mains. It would be interesting to know the age of this pipe, and the experience which the city of Detroit has had with steel pipe.

In determining the amount of water required in the future it would be interesting to know what results the author would have obtained had the Siems-Biser formula been applied. I am still convinced that this method of calculating future demands (Proceedings of New England Water Works Association, Vol. 43, No. 3, "Factors in the Design of Distribution Systems") is the most sound basis. Mr. Stephenson also states, or rather I infer from his statements, that he prefers a single large feeder to a looped system. It would be interesting for him to explain in discussion the reasons for his opinion in this connection.

The following statements from the above mentioned proceedings of the New England Water Works Association may be of interest due to the author's presumed viewpoint regarding single larger feeders:

"Storage and consequent pressure support, afforded by balancing reservoirs of standpipes, are important as parts of distribution systems. Balancing reservoirs, particularly at the ends of long feeder mains and at the extreme boundaries of service zones are automatic pressure adapters, resulting in uni-

[1] Executive Vice-President and Chief Engineer, National Water Works Corporation, New York, N. Y.

form performance of distribution systems under varying conditions of draft. The presence of these adapters makes possible the use of smaller feeders without increasing friction losses or adding other detriments to service. These reservoirs also provide reserve storage to carry over chance periods of pump breakdowns. For better distribution performance, however, it is recommended that storage be distributed among several small reservoirs rather than being concentrated in a single large balancing reservoir. Both balancing-reservoir storage and pumping-station capacity should be sufficient to serve adequately the system whose primary main sizes have been fixed by calculated maximum demands."

The sizes of pipe in the distribution gridiron seem small. It would be interesting to know in greater detail how the system is grid-ironed. I am of the opinion that the more economic design is represented by a major skeleton work of large feeders and the interior gridiron of small closely cross-connected mains. To make such design possible a sensible street lay-out is required, and from the author's statements such street lay-out seems to be the admirable practice of Detroit.

The St. Louis Water Department, by the statements in the paper by Skinker and Serkes, is planning ahead intelligently by availing itself of the individual studies of growth made by other utilities. Great losses have been experienced by large cities in the past because water supply systems have been permitted to grow like Topsies, improvements being provided from hand-to-mouth rather than in accordance with a comprehensive long-term plan of development.

The author in detailing the improvements to be made indicates the presence of feeder mains which had been installed with dead ends. It is very important to my mind to design and locate feeders that will be entirely useful, and that will be a complete link in the distribution system, because when large mains are installed piece-meal, it very often happens that before the entire feeder main has been completed the geographical location of the demand has been so altered that the main would have been better located in another place. This author favors the two ways system of water mains installation, and it would be of interest to have his reasons as against those of Mr. Stephenson.

J. B. EDDY:[2] A study of these two papers suggests many points which may be to the mutual advantage of all water works engineers, especially to those charged with the development of the distribution

[2] Engineer, Water Pipe Extension, Bureau of Water Supply, Chicago, Ill.

system. Only those considered most important will be touched upon in this discussion.

Various methods of making improvements are practiced by those charged with the responsibility of providing adequate facilities for distributing water. The most common is that of obtaining the services of a consulting engineer or engineers to work in conjunction with local officials. This method was used by Detroit. It has its merits in that an independent, unbiased viewpoint can be taken by the consultant. His judgment will usually be given more weight than that of a local or attached engineer.

St. Louis obtained the services of the Pitometer Company to make a survey and examination of their system which was used in connection with their distribution plan.

Chicago depends upon a small force of attached or staff engineers, who have had many years of service and who are charged with routine duties, in addition to planning future extensions to the system.

As is the case of Detroit and St. Louis, a twenty-five-year plan is made. This twenty-five-year plan, however, is revised every five years, United States Census years, and intermediate five-year periods.

The men who make the plans are attached throughout the construction of the work and supervise the improvements in service.

The first twenty-five-year plan was made to extend to 1940 and under this plan all extensions during recent years have been constructed.

Under the terms of the Sanitary District Act, which was passed by the Legislature of Illinois in 1889, Chicago must provide water for many suburban towns. Therefore, plans must be drawn from the metropolitan district. At present the city and suburbs supplied with water, have a combined population of 3,600,000 and an area of 290 square miles.

Data upon which the 25 year plan is based are as follows:

1. United States Census figures, past and projected into the future for each square mile. The square mile is the unit area used in all studies.

The census curves are supplemented by a map of the entire area upon which population is indicated by dots, each of which represents 500 people. This map indicates density of population and trend of growth better than curves or figures. It indicates very strikingly that population growth follows good transportation and water supply.

2. An occupation survey for each unit area is obtained from record atlasses of the Building Department and from a field inspection made every five years. This survey shows the area occupied in each square mile for all purposes and the vacant. The zoning ordinance is used to determine what class of occupation will be allowed in the vacant areas. The occupation survey sheet shows the amount of water used by industries, commercial houses, railroads, parks, and for domestic purposes, all of which is determined either by pitometer survey or by meter readings. The occupation survey also covers all outside towns receiving water from Chicago and other territory entitled to water under the Sanitary District Act.

3. Pitometer surveys are available for each square mile in the city proper and meter readings for all outside towns and other consumers which are metered at the City limits. These surveys are used to determine maximum and minimum usage. They also indicate sections where intensive survey work is necessary to eliminate waste and leakage.

4. Trunk line surveys are made yearly by pitometer which give accurate information concerning present operating conditions. Pump slip tests are made every five weeks.

5. Meter flow curves for all classes of consumers are available. From these the characteristic twenty-four-hour curve is determined and the time of ordinary maximun, as well as the maximum hour load. These curves are of great value in designing.

6. A study of all metered services in the City by classes has been made. This determines the amount of water required per service by each class without reference to per capita usage. This study is used to determine averages, while the twenty-four-hour meter curves are used to determine maxima and minima.

7. Fire flow tests are made in each quarter mile area at least once in five years, and personnel is furnished every ten years to the National Board of Fire Underwriters to assist in their decennial survey. The standards of the National Board are used in designing to meet requirements for fire flow. All fire flow tests are made by open hydrant methods, actual fire flow delivery being determined.

8. Information similar to that obtained by Detroit and St. Louis from utilities, railroads and consulting engineers is also given weight in planning extensions.

In designing extensions it is necessary, not only that the maximum hourly requirement be known, but that the time when it will occur

and the point or points of delivery be determined. The adoption of the square mile unit has made it possible to determine, not only the maximum hourly requirement, but the time when it will occur and the point at which the hourly requirement shall be applied.

Chicago is similar to Detroit in many respects. Rapid growth of industries and population have been the rule in both cities. The maximum elevation served in Chicago is about 90 feet above the lake level or city datum. A similar maximum elevation is found in Detroit. The use of elevated tanks has been given some consideration, but to fill such tanks in Chicago would require the laying of long isolated lines of large pipe or the maintenance of comparatively high pressure throughout the night on very large sections adjacent to, as well as at a distance from, the pumping stations which would prove very costly without system, as operated at present, that is, with only 20 percent of the services metered. Such tanks would be needed on a comparatively few days in the hot dry summer weather.

The twenty-five-year plans adopted for Detroit and St. Louis have been intelligently worked out and will undoubtedly serve the purpose for which they have been designed.

LOUIS E. AYRES:[3] The writer has read with great interest these two papers and welcomes this opportunity to discuss, briefly, some of the striking elements of similarity and of difference as between the two great water distribution systems described by the authors.

Both Detroit and St. Louis started their water works plants a little over 100 years ago, in 1825 and 1829 respectively, under the direction of private owners. Detroit drew its supply from the clear waters of the Great Lakes and St. Louis from the "Muddy Mississippi River." St. Louis distributed water through 2-, 3- and 4-inch mains, Detroit through tamarack logs bored out to $2\frac{1}{4}$- and 3-inch. The St. Louis Water Works was purchased by the City in 1835, the Detroit Water Works in 1836. From 1836 till 1930 there occurred, in each city, a series of expansions of plant and equipment, a moving upstream of the water works intakes, the introduction of filtration and the development of extensive distribution systems to meet the ever increasing demands of rapidly growing populations.

St. Louis preceded Detroit in the matter of filtration by nine years, building "in 1914 a modern filter plant of 160 million gallons

[3] Consulting Engineer, Ayres-Lewis-Norris and May, Ann Arbor, Mich.

capacity;" while Detroit "formally opened" on "December 22, 1923—the largest plant devoted to the purpose of clarifying and purifying water in the world," with "a filtering capacity of from 320 to 360 million gallons daily."

Both cities, during recent years, have undertaken comprehensive studies in anticipation of their respective water needs for the next quarter of a century. St. Louis is projecting its plant to supply a population within the city of 1,101,455 in 1955 and about 300,000 additional in the County of St. Louis; and Detroit is preparing for a population of over 3,500,000 in its metropolitan area in the decade 1950 to 1960. It is interesting to note the similarity of methods employed in the studies that have been made to anticipate the future. The cities were analyzed, in each case, by districts; use was made of all local population estimates including those of the United States census, of the local telephone company, and those made for rapid transit and other public improvements; and a pitometer and water waste survey together with extensive department data on the use of water including fire hydrant surveys afforded a basis for estimating the probable future consumption. The Detroit studies were incorporated in a report by Mr. George H. Fenkell, Superintendent and General Manager of the Department of Water Supply, dated September 15, 1923.

In each city the anticipated average use of water has been placed at about 150 gallons per capita per day. In Detroit the anticipated maximum day was originally placed at 1.3 times the average and the maximum hour at 1.6 times the average, but plant designs proceeded on the basis of 1.35 and 1.8 respectively. Subsequent studies, based on recent records, indicate that these ratios, with adequate supply, may run as high as 1.45 and 2.0. In St. Louis the corresponding figures were placed at 1.5 and 2.25 times the average, respectively.

Significant differences in the distribution plans, as worked out in the two cities, arise principally from two elements (*a*) topography and (*b*) size of the projects.

If the expression may be permitted, "What a grand and glorious feeling" it would be to those responsible for water distribution in Detroit, if, in some mysterious way, a hill or two could be raised to a height of 150 feet or more above surrounding territory, one to the north and another to the west of the city. Although the surface of the ground in Detroit rises gradually to the northwest, the highest point being about 90 feet above the lowest, there are no abrupt rises

affording sites for reservoirs. Lacking such a natural asset, Detroit has recently, at certain outlying points, resorted to the more expensive expedient of elevated steel tanks which, although useful in maintaining pressures during peak demands, contain but "a drop in the bucket" as compared to the total consumption for a day. When the pumps shut down in Detroit, therefore, the water supply ceases. On the other hand, in St. Louis high points are available for large storage reservoirs. The ultimate plan contemplates a storage of nearly 300 m.g. or in excess of a maximum day's consumption in 1955, which stored water may flow by gravity and provide adequate pressures in all parts of the system. This situation provides an element in the St. Louis system, making for continuity of service, with less responsibility on the pumping stations and less strain on the distribution system than in Detroit.

The Detroit System is not only much larger as a whole than the St. Louis System, but all of the elements are larger. Detroit plans to supply 3,500,000 persons or more, from a single intake, and three distributing centers. In St. Louis less than one-third the population will be served from two intakes, two filter plants and three pumping stations.

Detroit's filter plants will be at the distributing centers, the raw water being delivered thereto through large gravity tunnels, thence pumped, filtered and pumped again into the distribution system; the St. Louis filter plants are near the intakes, the filtered water being delivered through conduits, in the one service by gravity to a high lift pumping plant, and in the other by pumping to a large covered reservoir for gravity delivery to the distribution system.

The early studies in Detroit considered filtration for all additional supply at or near the source, but it was feared that the possible infiltration of ground water into tunnels, when out of service, might prove a source of contamination and the plan to carry raw water only through concrete tunnels in clay was, therefore, adopted. The bearing of this decision, if any, on the gravity conduits in St. Louis between Chain of Rocks Plant and Baden and Bissel's Pumping Stations, is unknown to the writer, and information thereon would be of interest.

The distribution of much larger quantities of water from distributing centers results in larger distributing mains in Detroit than in St. Louis. The largest distributing main in St. Louis, outside of the 5-foot conduit from the Stacy Park reservoir, appears to be 36 inches,

and four such mains is the largest number going out from any station. In Detroit the existing Water Works Park station delivers its supply into 13 mains varying in size from 42 to 60 inches in diameter. In the Springwells Station the water will be delivered initially into one 72- and two 66-inch mains. This situation in Detroit has obviously forced the use of steel pipe, although breakages in large cast iron mains and initial cost had led to the use of some steel several years ago. Two of the mains out of Water Works Park Station are steel, one 48 and one 60 inches in diameter. The general rule that has been adopted in Detroit is to use steel for diameters larger than 36 inches, and cast iron for 36 inches and smaller.

Both cities separate their distribution system into distinct service areas by closed gates. In St. Louis there are two such areas, in Detroit, three, plus a boosted area within the city, and eventually there will be four, plus two or more boosted areas beyond existing city limits. In St. Louis each service area has one pumping station and one reservoir, plus a "stand-by" station on the Stacy Park service; in Detroit each of the three principal service areas will be served by two pumping stations, the water from each ordinarily intermingling at about the mid-point between each pair of stations. Large loop mains connect all stations so that in the event of shut down or curtailment of supply from one station, the other, serving the same area, may assume a larger portion of their common load.

In St. Louis the average pressure maintained on the Compton Hill Service is 40 pounds, and on the Stacy Park Service 60 pounds. In Detroit pressures have been gradually increased with the years and in 1929 the mean pressures from the several gages varied over the following range:

Low service	22 to 40 pounds to 49 at pumps
Intermediate service	33 to 60 pounds to 65 at pumps
High service	24 to 48 pounds to 74 at pumps
Boosted area	43 to 51 pounds

In the contemplated Detroit plan, which will be inaugurated with the completion of the Springwells Station, pressures, at about the center of each service area, will be maintained as nearly constant as may be and will range at the several control points from about 40 to 60 pounds. The total range to consumers over a service area from minimum to maximum hour will vary from perhaps 70 pounds near the station to 25 pounds at the extreme limits of the area. Maxi-

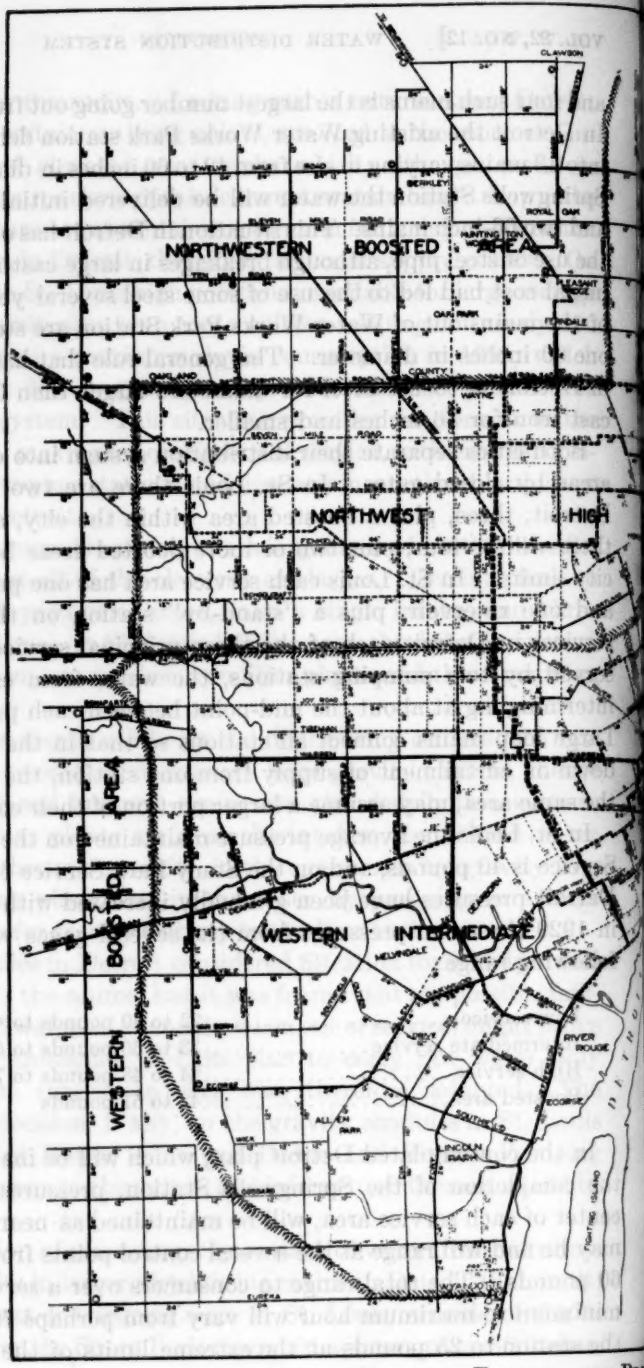
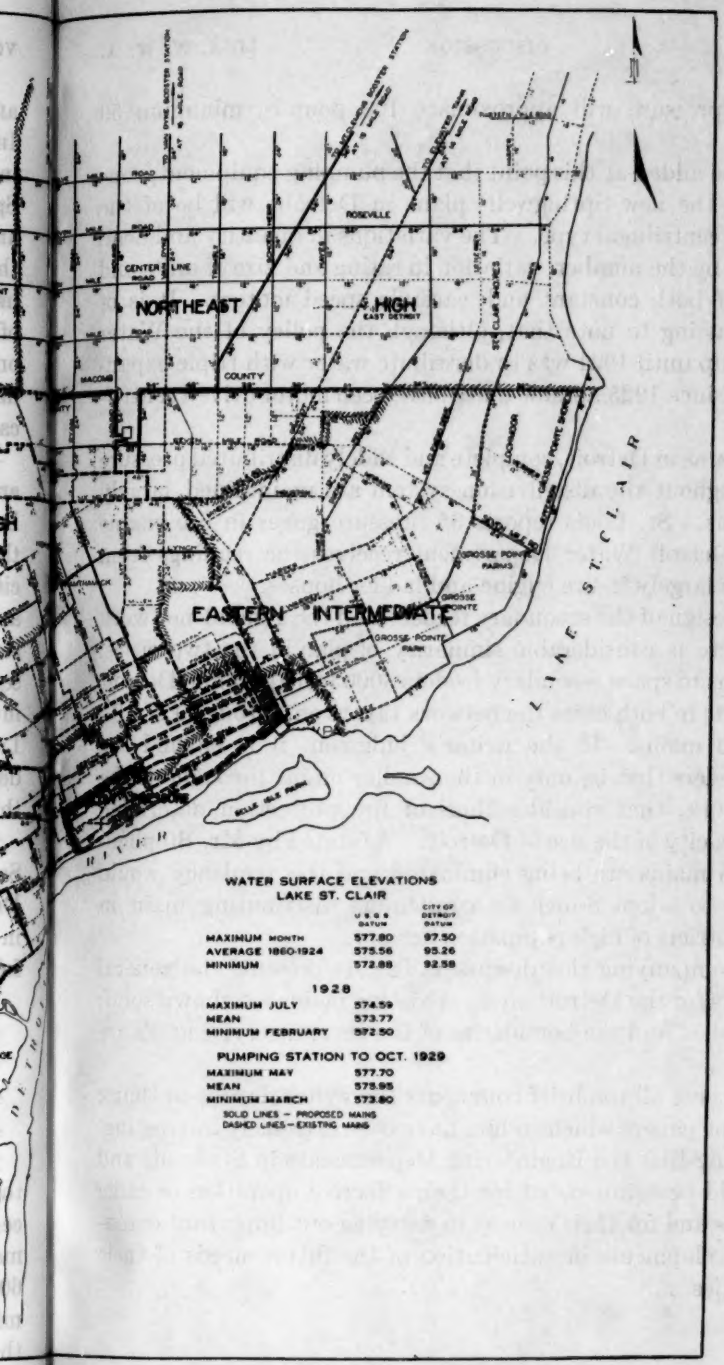


FIG. 1. MAP OF PRINCIPAL EXISTING AND PR ATER M



D PR ATER MAINS FOR A DETROIT METROPOLITAN AREA

mum pump pressure will approximate 100 pounds, minimum 50 pounds.

It should be added at this point that the pumping equipment, contemplated in the new Springwells plant in Detroit, will be of the motor driven centrifugal type. The variations in quantity and head are to be met by the number, variation in rating and size of units and by the use of both constant and variable speed motors. It is of interest in passing to note that, although the policy of the Water Department up until 1924 was to distribute water with triple expansion engines, since 1925 all new units have been motor driven centrifugals.

In St. Louis, as in Detroit, complete and widely distributed pressure records throughout the distribution system are maintained, largely in fire stations. St. Louis reports 35 pressure gages in fire engine houses; the Detroit Water Department receives the readings from 74 gages, also largely in fire engine and ladder houses.

As to the design of the secondary feeders and the general net work of mains, there is considerable similarity of plan in the two cities. St. Louis plans to space secondary feeders 3000 feet apart and Detroit one mile apart; in both cases the network tapers off through 12-, 10-, 8- and 6-inch mains. In the writer's judgment it is beyond the secondary feeders that is, only in the smaller mains throughout the general network, that considerations of fire protection determine main sizes in a city of the size of Detroit. As stated by Mr. Stephenson all 4-inch mains are being eliminated and the tendency would appear to be to adopt 8-inch as a minimum distributing main in residential districts of high population density.

A map accompanying this discussion (fig. 1) presents the general plan of feeders for the Detroit area. Existing mains are shown solid; proposed, dashed and the boundaries of the several service areas are indicated.

In closing these all too brief comments the writer desires to thank the authors for papers which to him have been extremely interesting. It is also fitting that the Engineering Departments in St. Louis and Detroit should be commended for their effective operation of large water systems and for their success in carrying out important extensions and developments in anticipation of the future needs of their respective cities.

WATER SOFTENING AND PURIFICATION METHODS[1]

By J. A. McGarigle[2]

Successful clarification of turbid, or colored, waters is generally dependent on proper coagulation preceding filtration. If this coagulation is neglected, the best designed and operated filter equipment will fail to deliver an effluent of desired clarity. Much emphasis has frequently been misplaced on the filters in a plant, equipping them with expensive rate-of-flow controllers, loss-of-head gauges and other control devices, but failing to make proper provisions for coagulant feeds, agitation and settling. This is particularly true in the case of industrial installations.

The importance of mechanical agitation of the water directly after the addition of the coagulants, has begun to be appreciated in the latest municipal plants, but it has generally been assumed that such agitation is not warranted for the average industrial installation. Experience has shown, however, that even for industrial plants of moderate size the investment in such mechanical agitators is amply repaid by the improvement in results and by the savings realized. Where the floc forms slowly, a settling tank of four to six hours' detention period may be required if mechanical agitators are omitted. But if they are provided this detention period may frequently be reduced to two hours or less. These agitators hasten the formation of the floc and also give the floc greater stiffness so that the mat that forms on the filter beds is not easily broken. Furthermore, the better flocculation of the water causes a greater reduction of the turbidity and color in the settling tank, thus relieving the filters of a considerable burden. This lengthens the filter runs between washings and thereby reduces the labor of attention and percent of wash water required.

Some surface waters of low turbidity and high color at certain seasons of the year prove quite resistant to decolorization. The floc forms slowly and imperfectly even with very long periods of

[1] Presented before the Toronto Convention, June 28, 1929.

[2] District Manager, Permutit Company of Canada, Ltd., Toronto, Can.

detention. High amounts of coagulant are frequently used to hasten the flocculation. By using a good mechanical agitator, together with the addition in some cases of clay, the floc forms rapidly and the decolorization is readily accomplished. Furthermore, relatively small amounts of coagulant are adequate, thus effecting a marked economy in operating cost for chemicals.

The settling tank may be of steel or concrete, of vertical or horizontal design. The use of the vertical steel cylindrical tank has many advantages over the horizontal type.

a. It is usually less expensive for moderate size industrial installations.
b. It occupies much less floor space.
c. It permits easy means for desludging because the bottom of the tank is above the sewer level and a sludge spider inside the tank can be operated while the tank is full. In this way, the draining of the tank and manual removal of the sludge is avoided.
d. Settled water in the top of this tank may be used as a gravity wash-water supply for the filters, thus saving the investment in and operation of wash pumps.
e. The raw water supply frequently is used under pressure for other purposes in the industrial plant. This pressure is dissipated if the water is introduced into a concrete horizontal basin below ground. With the vertical steel settling tank, the pressure is utilized to reach the top of the tank, and the head of water in the tank saves considerable power for pumping through the filters to service.

WATER SOFTENING FOR BOILER FEED

Marked changes in boiler design have taken place during the last few years. Boiler pressures have increased appreciably. Boiler ratings have gone up so that more water is evaporated per square foot of heating surface than formerly. Pulverized fuel is becoming popular and water-cooled furnace walls are being generally adopted. These changes have in turn resulted in a demand for a water of the lowest possible hardness so that no scale is formed. The thinnest film of scale can no longer be tolerated, because it results in overheating of the metal, causing failure of tubes by blistering, etc.

To meet these demands, three methods of water treatment are being applied:

1. Evaporators, or a combination of evaporators and zeolite.
2. Hot lime soda.
3. Zeolite.

EVAPORATORS

The use of evaporators is generally confined to the central power station field inasmuch as only a small percent of makeup can be

TABLE 1

Summary of evaporative make-up costs per 1,000 pounds

	FIXED CHARGES, CENTS PER 1,000 LB. EVAP.	OPERATING CHARGES, CENTS PER 1,000 LB. EVAP.	FUEL CHARGES, CENTS PER 1,000 LB. EVAP.	COST OF CHEMICALS	TOTAL
Edison Electric Illuminating Co.	7.00	Indeterminate	Indeterminate		7.00
Northern States Power Co.	5.37	2.25	2.00	0.66	10.28
Ohio Power Co.		6.46	0.6		7.06
Oklahoma Gas & Elec. Co.	2.78	1.50	0.49		4.77
Pennsylvania-Ohio Power & Light Co.	1.50	1.18	0.0193		2.873
Detroit Edison Co.	1.70	0.6	Not given		2.30
Duquesne Light Co.	5.79	0.852			6.642
Commonwealth Power Corp., Michigan	3.86	2.57	7.19		13.62
Average per 1,000 pounds, cents					6.82

The average cost of evaporating the water is thus 6.8 cents per 1,000 lb. or over *55 cents per 1,000 gallons*. In the case of hot lime soda or zeolite for an average water of 10 gr. per gal. hardness, the average cost would be well under 5 cents per 1,000 gallons, allowing for fixed charges, chemicals, labor, etc.

economically treated in this way. Where the percent of makeup becomes appreciable, e.g., over 5 to 10, the first cost of the evaporator installation, as well as the cost of operation, becomes very high. Even with these small make-up requirements the cost of distillation is many times higher than either of the other two methods mentioned. The National Electric Light Association in the report of their Prime Movers Committee on "Treatment of Feed Water," 1927–1928, page 3, gave a table of evaporative make-up costs. (See table 1.)

Distilled water, moreover, tends to cause corrosion and therefore alkali must be added to the distilled water. This in turn causes a low sulphate-carbonate ratio and therefore, sodium sulphate is then added to build this ratio up. On page 2 of this report this procedure is described as follows:

"Due possibly to the amount of dissociation of water (distilled) at the saturation temperature (about 440°F.) it is necessary to treat the water chemically to prevent corrosion. . . . . When the phenol alkalinity is below 25 p.p.m. or the pH value is below 8.8 enough soda ash is added to the boiler water to bring it to this point. . . . .

"Glauber's salts are also added to the boiler water to keep the ratio of sodium sulphate to sodium carbonate above 3.5 in order to prevent caustic embrittlement."

Thus the raw water is first distilled in an attempt to have water free from all solids and then dissolved solids are reintroduced.

ZEOLITE EVAPORATOR COMBINATION

Where evaporators are fed with raw water reliance has sometimes been placed on cracking the resulting scale from the coils of the evaporator. This is possible with certain types of brittle scale, but the trend is away from this crude method of keeping evaporators clean. The cracking method causes the coils themselves to suffer so that high maintenance costs result. Furthermore, the scale affects the capacity of the evaporators. Also when they foam, or prime, the raw water is carried over by the vapor so that hardness is present in the distilled feedwater. Since the first cost of zeolite plants to soften the makeup to the evaporators is but a small percent of the cost of the total evaporator installation, a number of evaporator users have added such zeolite pretreatment. The Prime Movers Committee of the National Electric Light Association in their report on "Treatment of Feed Water" 1928–1929 (April, 1929) (Publication 289–53) on page 1 states:

"Of the 24 plants using evaporators, ten pretreat the evaporator feed. Of these ten, six use zeolite softeners while one employs a unit system of evaporators and has a zeolite softener installed ahead of only one of these four units."

A comparison of the capacity of evaporators receiving water pretreated by lime and alum and an evaporator receiving water pretreated by zeolite is given on page 2 of this report as follows:

"Ohio River water treated with lime and alum to remove suspended, organic matter and bacteria is fed to evaporators 1, 3 and 4, but the water for unit No. 2 is also treated by Permutit Water Softener.

"The average output of evaporators 1, 3 and 4 is 5,500 to 6,500 lb. per hr. whereas unit No. 2 delivers about 7,500 lb. per hour.

"Evaporators 1, 3 and 4 are 'cracked' with cold water every day and manually cleaned every 6 to 8 weeks, requiring about 27 manual hours for each cleaning. Unit No. 2 does not require these cleaning methods."

Another user of the zeolite evaporator combination in the 1927–1928 report of the N. E. L. A. on pages 15 and 16, gave their experiences as follows:

"The zeolite system at the Long Beach plant No. 2 of the Southern California Edison Co. was installed for the purpose of removing scale-forming constituents from the evaporator feed water. Prior to the installation of the zeolite system, considerable trouble had been experienced due to the formation of scale on the evaporator tubes. This scale was hard and would not crack off, hence frequent cleanings were necessary, causing low evaporator operating efficiency and capacity.

"The evaporators were cleaned once a month by chipping the scale off the tubes by hand. This required 1½ days per evaporator at a cost of $100 or $400 per month for the complete installation.

"Acid treating was resorted to at a later date and was used for a period of about 4 months.

"A clean evaporator will evaporate 9,000 lb. of water per hour. The average evaporation from one cleaning to the next was 6,052 lb. per hour or 67 percent of the maximum. In addition to this, there is also the loss in capacity, due to the evaporator being out of service during cleaning which amounts to about 1 percent for the acid cleaning and 5 percent for mechanical cleaning. . . . .

"Since the installation of this zeolite system over a year ago, no cleaning of the evaporators has been necessary, and the evaporator capacity has been maintained at maximum rating at all times."

HOT LIME SODA AND SULPHATE-CARBONATE RATIO

It is possible with hot lime soda treatment to reduce the hardness to 1.5 grains per gallon as $CaCO_3$ with an excess of soda ash of about 2 gr. per gal. This hardness is not low enough to prevent troubles from scale and tube failures in high pressure boilers. These troubles are particularly emphasized by the hard scale that results if high sulphate-carbonate ratios are maintained as suggested to inhibit corrosion fatigue or so-called embrittlement.

The sulphate-carbonate ratios suggested in boiler water to inhibit so-called embrittlement were previously:

RATIO Na_2SO_4/Na_2CO_3	BOILER PRESSURE (LB./SQ. IN.)
1:1	Up to 150
2:1	150–250
3:1	Over 250

In the May, 1929, issue of "Mechanical Engineering" the A. S. M. E. Committee on "Care of Power Boilers" now suggests[3] increasing this ratio to the following:

RATIO	BOILER PRESSURE (LB./SQ. IN.)
2:1	Under 150
0.014 × Boiler Pressure	Over 150

The Committee has therefore raised the minimum ratio from 1:1 up to 2:1 and for higher pressures, the ratio is also much higher than previously suggested. Due to the excess of soda ash employed in hot lime soda plants to obtain as soft an effluent as possible to prevent scale formation, even the older ratios were difficult to maintain especially where the sulphates in the original water were low. With the increased sulphate-carbonate ratios it will be still more difficult and therefore the user of hot lime soda plants will have to adopt independent measures of building up this ratio. This usually consists of adding sodium sulphate. But for a boiler pressure for example of 400 lb. the ratio suggested is $400 \times 0.014 = 5.6$.

With an excess of soda ash of 2 gr./gal. or 34 p.p.m. the sodium sulphate content required will be

$$34 \times 5.6 = 190 \text{ p.p.m.}$$

If the sulphates in the original water are 30 p.p.m., for example, this means the addition of 160 p.p.m. or $1\frac{1}{3}$ pounds of anhydrous sodium sulphate per thousand gallons. This not only adds appreciably to the operating cost, but greatly increases the total dissolved solids, thus in turn increasing the required boiler blow-off to prevent foaming of the boilers.

[3] This suggestion to raise the ratio has not yet been and may not be adopted by the Committee.

Furthermore with 1½ grains of hardness present and a high sulphate-carbonate ratio there is great danger of calcium sulphate scale forming.

In the N. E. L. A. "Feedwater Treatment" of July, 1926, page 4, this danger is very well described by an actual experience:

"The following is a statement of a member company with boiler-scale troubles, following an attempt to maintain the suggested sulphate-carbonate ratio in boilers operating at 350 lb. pressure.

"The boiler feed water at this station consisted of approximately 97 percent condensate returns and 3 percent make-up. The boiler feed makeup was supplied by evaporators. The water being evaporated was slightly alkaline and at the same time carried considerable carbonates of calcium and magnesium. In increasing the concentration of water in the evaporators, priming carried over solids with the vapor. Condenser leakage was at all times held down to less than 0.5 percent. Condenser leakage also brought dissolved salts into the boiler-feed-water system.

"The boiler feed water had an adverse sulphate-carbonate ratio when considered from the viewpoint of caustic embrittlement. . . . . It was believed that the simplest method would be to introduce the proper amount of sodium sulphate directly into the feed water in order to maintain the proper ratio of sulphates to carbonates.

"After proceeding in this manner of treating the water for a short period of time, tubes began to fail in the bottom rows of the boilers. The scale was analysed and found to be calcium sulphate scale. . . . . In other words, the practice recommended by the boiler manufacturer developed a water condition which caused scale formation. This was due to the fact that a large proportion of the sodium carbonate in the boiler water was broken down into sodium hydrate at the operating pressure."

In other words, in the absence of sufficient carbonate radical, the sulphate ions combined with the calcium present in the water and calcium sulphate scale deposited on the evaporating surfaces.

This N. E. L. A. statement furthermore refers to another experience, last paragraph on page 4 of the 1926 report:

"It may be of further interest to note that some difficulty was experienced at another station when attempting to maintain the caustic embrittlement ratio, due to the fact that the OH concentration in the boiler drum water was being increased to such an extent that calcium hydrate instead of calcium carbonate was precipitated. The hydrate came from the decomposition of sodium carbonate in the boiler water. The trouble did not become serious at this station, due to the fact that the make-up water had much less calcium and magnesium salts than the water evaporated at the other station first mentioned above."

These experiences clearly indicate the troubles that the user must face if he endeavors to increase the sulphate content in the effluent of a hot-lime soda plant. There is sufficient calcium present in this water so that in the presence of a high amount of sulphate ions and a low amount of carbonate ions, a hard calcium sulphate scale forms. In this regard it may be of interest to refer to the paper by E. P. Partridge and A. H. White entitled, "The Formation and Thermal Effects of Calcium Sulphate Boiler Scale," which was read at the Rochester meeting of the A. S. M. E. in May, 1929. On page 6 of this paper the authors state:

"In studying the heat effects of scale in boilers of modern design, emphasis must be shifted from the viewpoint of heat utilization to that of overheating of the boiler metal. It is precisely the heat transfer surfaces exposed to the most severe conditions on which calcium sulphate or silicate scale will form most rapidly. While scale deposits in the front row boiler tubes can have no effect upon the absorption of radiant energy by the tube, it does offer, as it grows in thickness, a steadily increasing resistance to the flow of heat from the tube metal to the liquid within the tube. As a consequence the temperature of the tube metal must increase in order to drive through the increased resistance the constant flow of heat it is receiving by radiation. Not a very great amount of scale is necessary to raise the tube temperature to the point at which the steel will fail."

HOT LIME SODA PLUS PHOSPHATE

More recently, however, the use of phosphate has been suggested as an alternative method to sulphate-carbonate ratio for inhibiting corrosion fatigue (see Straub's article in Power Plant Engineering, February 1, 1929, page 173). For this reason, the use of phosphate is now frequently resorted to with hot lime soda plants in an effort to avoid the hard calcium sulphate scale that results when hot lime soda is used together with a sulphate-carbonate ratio. J. D. Yoder (The Journal of the Amer. Water Works Association, Vol. 21, No. 1, Jan., 1929), states:

"However, since having the assurance of Professors Parr and Straub that the continual presence of a small amount of tri-sodium phosphate in the boiler will prevent embrittlement even though the sodium carbonate may be high with respect to sodium sulphate, a considerable advantage is given to the use of tri-sodium phosphate for the high pressure boilers. The writer, therefore, desires to state in support of Dr. Hall's advocacy of phosphate for high pressure boilers, that it is now the policy of the Cochrane Corporation to advocate that the hot process lime and soda treatment should be supplemented with tri-sodium phosphate for boiler pressures of 350 lb. per sq. in. and higher. . . . .

The use of phosphate in this way adds considerably to the operating cost, because all the hardness present in the feed water must first be precipitated before any soluble PO_4 is left in the boiler salines to act as a so-called inhibitor, and the amount of phosphate thus required to react with the lime present is many times that amount recommended to be kept in solution. The following calculation shows this:

Amount of phosphate to precipitate hardness of hot lime soda. Assume the hot lime soda effluent to have a hardness at best of 1.5 grams per gallon (27 p.p.m.) as $CaCO_3$. Of this about 1.0 gr./gal. is calcium. Each grain of calcium requires about 2.5 gr./gal. of disodium phosphate ($NaHPO_4 \cdot 12H_2O$) to precipitate it. The cost of this is:

2.5 gr./gal. = 2.5/7 lb./M gals. = 0.36 lb. at 4c = 1.44c per M gals.

Amount phosphate to form soluble PO_4 in boiler salines. Straub's rule (see Power Plant Engineering, Feb. 1, 1929, page 173).

BOILER PRESSURE, LB./SQ. IN.	MINIMUM PO_4 IN SOLUTION IN SALINES
Under 250 lb.	40 p.p.m.
Over 250 lb.	40 p.p.m. for alk. under 1,300 p.p.m. Add 20 p.p.m. for each 400 p.p.m. alk. over 1,300 p.p.m.

The amount of phosphate required by this rule is usually quite small, amounting to 1/8 to 1/12 lb. per M. gals. after all the Ca is precipitated. 1/10 lb. at 4c = 0.4c per M. gals.

Summary (hot lime soda).—

Phosphate to precipitate hardness	= 1.44c per M. gals.
Phosphate to form soluble PO_4	= 0.4
Total	1.84c per M. gals.

WATER SOFTENING BY ZEOLITE

With the increasing pressure and boiler ratings, zeolite treatment has gained in favor for boiler feed water purification. Table 2 shows some typical recent zeolite installations for high pressure boilers.

Due to the increasing tendency to demand either a high sulphate carbonate ratio or the addition of a definite amount of phosphate ions as PO_4 in solution in the boiler salines, waters which previously

were considered favorable for hot lime soda treatment are now treated more economically, and with better results, by zeolite plus the secondary treatment for the ratio control.

The following is a typical comparison of treatment by hot-lime soda and zeolite of Lake Michigan water, which has a low sulphate-carbonate ratio in its natural state. Table 3 gives the composition of this Lake Michigan water after hot-lime soda treatment plus

TABLE 2

Recent zeolite installation for high-pressure boilers

INSTALLATION	NUMBER OF BOILERS	TYPE BOILER	H.P.	PER CENT MAKE-UP	PER CENT RATING	POUNDS STEAM PRESSURE
1	4	Kidwell	1,000	33	300	250
2	2		1,000	30	350	450
3	2	Erie City	807	50	225	250
	5	Waste heat	450	50	78	250
4	3	Heine	600	30	200	400
5	3	Heine	1,000	28	300	450
	4	B & W	400			175
	2	Erie	500			175
6	3	Crossdrum	1,000	50–100	375–650	450
7	6	Bigelow Hornsby	1,200	6.0		300
8	3	B & W	1,750		200–400	400
9	8	B & W	1,500			400
10	7	B & W	1,500	4		300
11	1	Combustion Engineering		15		450
12	1	Combustion Engineering	6,000		150	450
13	3	B & W	1,291	25–45	225	275
	2	W. W.	1,291	25–45	350	275
	2	Comb. Eng.	150,000 lb. hr. each	25–45		275

sodium sulphate, or plus phosphate, and on the right-hand side, zeolite treatment plus sulphuric acid or phosphate. Table 4 shows the cost of these various forms of treatment per million gallons of water. Figure 1 is a graphic analysis of these comparative results and the treatment costs.

It will be noted in table 4 that the cost of treatment for the chemicals in the case of hot-lime soda is $17.30 per million gallons and the cost of sodium sulphate for ratio adjustment is $8.60, whereas the

cost of phosphate would be $13 per million gallons. In the case of zeolite it is $11.66 per million gallons for the chemicals and $4.30 for ratio adjustment, or $3.25 for phosphate treatment. The use of sodium sulphate is more economical in the case of hot lime soda, and the total for lime soda and sulphate would be $25.95 per million gallons. For the zeolite treatment the most economical addition would be the phosphate, since salt, alum and phosphate would cost $14.91 per million gallons. The use of sulphate in connection with

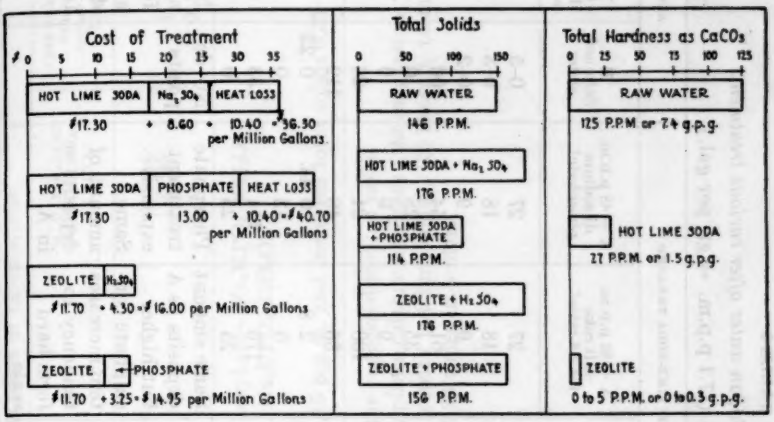


FIG. 1. GRAPHIC COMPARISON OF HOT LIME SODA AND ZEOLITE ON LAKE MICHIGAN WATER

Cost of treatment. In this cost should be included the chemicals for either ratio adjustment or phosphate addition as well as any heat losses occurring in the treatment. Heat loss shown herewith is based on a temperature drop through heater softener of 5°F., and figuring in terms of coal at $4.50 per ton, 9,000 B.t.u. effective per lb. coal.

Total solids. The total solids influence the boiler blowoff required. Where there are no suspensions present in the boiler salines the concentration can be carried at higher figures.

Hardness. The hardness in feedwater determines the amount of scale and sludge in the boilers. Each p.p.m. represents 1 lb. deposit per million lb. make-up water.

hot lime soda, as above explained, however, involves the danger of calcium sulphate scale resulting from this combination, and the ensuing consequence of possible tube failure.

Reference has frequently been made to the lower total dissolved solids which hot lime soda treatment leaves in the treated water as compared with the total solids which zeolite treatment leaves in the treated water. Table 3 shows what may be expected in this regard in the case of a water like Lake Michigan. The treatment with

TABLE 3

Composition of Lake Michigan water after various treatments

(Results in p.p.m.) (17.1 p.p.m. = gr. per gal.)

	RAW WATER	HOT LIME-SODA TREATED			ALUM-ZEOLITE TREATED		
		A Softened water	A + 83 p.p.m. salt cake for 2.8 ratio*	A + 48 p.p.m. di-sodium phosphate†	B Softened water	B Treated with 69 p.p.m. 66° Be. Sulphuric acid for ratio*	B + 12 p.p.m. di-sodium phosphate†
Total hardness as $CaCO_3$	125	27	27	27	0–5	0–5	0–5
Calcium hardness as $CaCO_3$	75	18	18	18	0–3	0–3	0–3
Magnesium hardness as $CaCO_3$	50	9	9	9	0–2	0–2	0–2
M.O. alkalinity as $CaCO_3$	110	61	61	54	106	40	104
Phenol. alkalinity as $CaCO_3$	0	35	35	28	0	0	0
Chlorides as CaCl	9	9	9	9	9	9	9
Sulphates as Na_2SO_4	21	21	100	21	26	119	26
Sodium alkalinity as Na_2CO_3		36	36	48	112	42	110
Sulphate-carbonate ratio		0.58	2.8	0.44	0.23	2.8	0.24
Total phosphates as PO_4	0	0	0	13	0	0	3
Total solids	146	97	176	114	153	176	156
Incrusting solids	123	25	25	25	0	0	0
Results in boilers	Scale and sludge	200 lbs. insoluble deposits per million gals. feedwater	Same amount deposits as A but higher sulphate content increases tendency to form hard scale	Phosphate treatment expensive. Same amount of deposits as in A	No deposits	No incrustants therefore higher sulphates do not affect results	Phosphate treatment cheap because water is practically free from hardness

ness

Figures used as basis of calculations

	M. O. ALKALINITY REDUCTION AS $CaCO_3$	PARTS PER MILLION SULPHATE INCREASE AS Na_2SO_4	TOTAL SOLIDS INCREASE
(A) For adjusting sulphate-carbonate ratio:			
1 p.p.m. sulphuric acid (66° Be. = 93% H_2SO_4)	0.95	1.35	0.35
1 p.p.m. salt cake (95% = Na_2SO_4)	0	0.95	1.0
(B) For using phosphate as inhibiting agent (di-sodium phosphate—$Na_2HPO_4 \cdot 12H_2O$)			
1 p.p.m. Ca hardness as $CaCO_3$ consumes 2.4 p.p.m. $Na_2HPO_4 \cdot 12H_2O$.			
1 p.p.m. PO_4 corresponds to 3.8 $Na_2HPO_4 \cdot 12H_2O$.			
1 p.p.m. $Na_2HPO_4 \cdot 12H_2O$ reduces M.O. alkalinity 0.15 p.p.m. and increases total solids 0.33 p.p.m.			

* For boiler pressure of 200 lb. per sq. in. according to the suggested revised ratio.

† This phosphate consists of two parts: (a) first, to precipitate calcium hardness (the magnesium present may also consume phosphate increasing the phosphate requirement especially with hot lime-soda), (b) second, to maintain 40 p.p.m. PO_4 in solution in salines (1.3 p.p.m. in feed water concentrated 30 times in boiler).

sodium sulphate, for ratio adjustment, increases the total solids in the lime soda effluent so that it is about the same as after zeolite treatment and ratio adjustment. When phosphate is used there is a lower total solids in the hot lime soda effluent than after zeolite. On the other hand, the hot lime soda treatment produces suspended matter in the boiler salines due to the precipitation of the hardness. This suspended matter has a provoking effect on foaming, as confirmed by the experimental research work of Prof. C. W. Foulk, of Ohio State University (Industrial and Engineering Chemistry, No-

TABLE 4

Costs of chemicals per million gallons to treat Lake Michigan water by methods as in table 3

	LB. CHEMICAL	HOT LIME COST SOFTENING	SODA TREATED RATIO ADJUSTMENT	PHOSPHATE	LB. CHEMICAL	ALUM ZEOLITE COST SOFTENING	TREATED RATIO ADJUSTMENT	PHOSPHATE
cents per lb.								
Hydrate of lime.....0.6	1,300	$7.80						
Soda ash.....1.9	500	9.50						
Alum.....1.4					40	$0.56		
Salt.....0.3					3,700	11.10		
Salt cake.....1.25	690		$8.60					
Sulphuric acid (66°Be.)*.0.75					575		$4.30	
Di-sodium phosphate.....3.25	400			$13.00	100			$3.25
Chemical cost per million gals.		17.30	8.60	13.00		11.66	4.30	3.25
Heat loss 5°F. temp. drop.....		10.40						

(Note: 120 p.p.m. = 7 gr./gal. = 1,000 lbs. per million gallons.)

* This cost is based on tank car contracts.

vember, 1924, page 1121). C. H. Koyl, who has been chief chemist for Western Railroads for a number of years, has also confirmed this by large-scale experiments in which he shows the marked effect of suspended solids in provoking foaming. (Railroad Gazette, June 13, 1902, page 423.)

The Consolidated Gas Electric Light and Power Co., Baltimore, Md. contributed a statement of their experience with their zeolite system in the N. E. L. A. report of July, 1924, Publication 24–69. Figures 2 and 3, taken from this report, show what the boiler blow-

off was before the zeolite plant was installed and how much it had to be increased just immediately after the start of the water softener, due to the fact that the soft water loosened up old scale and the sus-

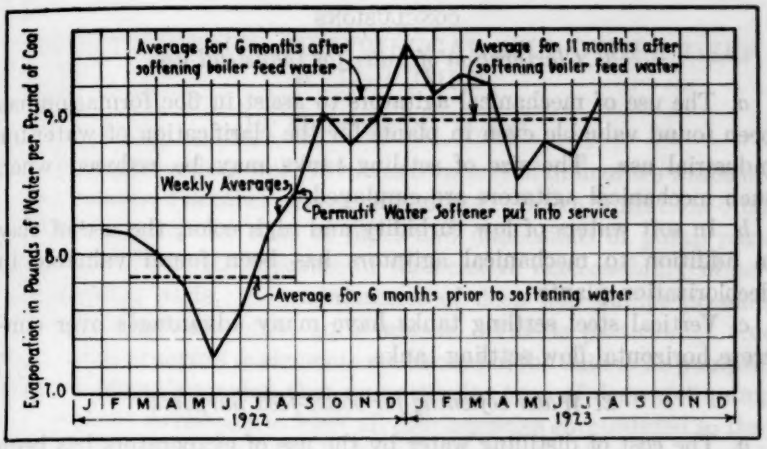

Evaporation in Pounds of Water per Pound of Coal
9.0
8.0
7.0
Average for 6 months after softening boiler feed water
Average for 11 months after softening boiler feed water
Weekly Averages
Permutit Water Softener put into service
Average for 6 months prior to softening water
J F M A M J J A S O N D J F M A M J J A S O N D
1922
1923

FIG. 2. EVAPORATION AFTER SOFTENING BOILER WATER

(N. E. L. A. Report, July, 1924, Publication 24-69. Spring Gardens Plant, Baltimore)

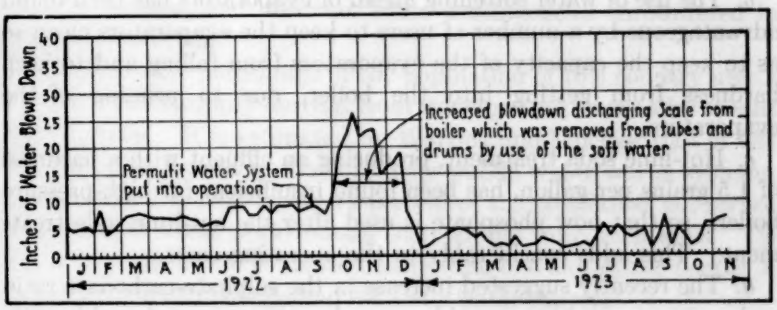

Inches of Water Blown Down
40
35
30
25
20
15
10
5
0
Increased blowdown discharging scale from boiler which was removed from tubes and drums by use of the soft water
Permutit Water System put into operation
J F M A M J J A S O N D J F M A M J J A S O N
1922
1923

FIG. 3. NUMBER OF INCHES OF WATER BLOWN DOWN PER BOILER PER DAY. (AVERAGE FOR WEEKLY PERIODS)

(N. E. L. A. Report, July, 1924, Publication 24-69. Spring Gardens Plant, Baltimore)

pended scale particles required increased blow-off to prevent wet steam. After these particles were removed, the curve of blow-off takes a rapid drop, and after January, 1923, the blow-off used was

lower than with the raw water. Figure 2 refers to the evaporation and shows how the evaporation increased as this scale came off. Figure 3 gives the blow-off at corresponding times.

CONCLUSIONS

1. Clarification of water for industrial use

a. The use of mechanical agitators to assist in floc formation has been found valuable even in plants for the clarification of water for industrial use. The size of settling tanks may be reduced when such mechanical agitators are employed.

b. In soft waters of low turbidity and high color, the use of clay in addition to mechanical agitators has been found valuable in decolorization plants.

c. Vertical steel settling tanks have many advantages over concrete horizontal flow settling tanks.

2. Water softening for boiler-feed purposes

a. The cost of distilling water by the use of evaporators has been found in the case of a number of users to be an average of 55 cents per thousand gallons, whereas the cost of treatment by hot-lime soda or zeolite would be under 5 cents per thousand gallons.

b. The use of water softening ahead of evaporators has been found advantageous by a number of users to keep the evaporators clean so as to keep the capacity of the evaporators from falling and to keep hardness from getting into the boiler, due to priming of the evaporators.

c. Hot-lime soda treatment, producing an effluent with a hardness of 1.5 grains per gallon, has been found insufficient for high-pressure boilers, so that now phosphate is used after the hot-lime soda treatment. This adds considerably to the operating costs.

d. The recently suggested increase in the sulphate-carbonate ratio makes it necessary to provide secondary treatment for this ratio adjustment with hot-lime soda plants. The increase in this ratio after hot-lime soda treatment involves the danger of production of hard calcium sulphate scale, which has frequently been found to cause overheating of boiler tubes, etc.

e. A comparison of zeolite and hot-lime soda treatment on a typical water like Lake Michigan shows that zeolite treatment has lower operating costs, lower hardness in the effluent and avoids the suspension in the boiler salines which are present in the case of hot-lime soda.

AIR LIFT VERSUS CENTRIFUGAL AND OTHER DEEP WELL PUMPS[1]

By Clayton N. Ward[2]

Pumping from deep wells involves a variety of difficulties which are not encountered in other classes of pumping. Space limitations often seriously restrict the sizes of pipes and dimensions of pump parts which may be used. Because of the relative inaccessibility of certain working parts, the maintenance of deep well pumps presents problems which are often quite serious. This is particularly true where sand or corrosive elements are carried in the fluid to be pumped. It is the writer's opinion that no particular type of deep well pump has yet been developed to meet all contingencies encountered so that it could be recommended to the exclusion of all other types for all requirements. This paper presents results secured from tests of various deep well pumps now commonly used, points out factors which favor or disfavor certain types, discusses considerations to be made in selecting pumps, and describes some contingencies encountered in deep well pumping.

A considerable percentage of the population of this country uses a domestic water supply derived from underground sources by means of deep wells. It is estimated by Warrick and Muegge[3] that of 283 municipalities served in Wisconsin by public water works systems, 212 use drilled wells. About 75 percent of the population not served by municipal water works obtain their supply from deep wells. On the basis of the above estimates, more than one-half of the people of Wisconsin use an underground domestic water supply.

TYPES OF DEEP WELL PUMPS IN COMMON USE

The reciprocating bucket pump, the turbine pump, and the air lift pump are the three types of equipment most commonly used in deep wells. The bucket or lift pump is not only the oldest type of

[1] Presented before the St. Louis Convention, June 5, 1930.

[2] With Mead and Seastone, Consulting Engineers, Madison, Wis.

[3] Safeguarding Wisconsin's Water Supply, This Journal, April, 1930, page 516.

deep well pump, but is probably the first reciprocating pump to be developed. In recent years, various types of motion have been applied to the pump heads with a view of improving conditions over that attainable with the simple crank and connecting rod. The general principles of air lift pumping have been understood for a great many years. The air lift pump was invented in 1797 by a German mining engineer, and came into practical use about fifty years later. Since 1880 several patents have been granted in this country on particular features of the air lift pump, and its use in certain districts has become quite common. The deep well turbine pump has been perfected so that it is a valuable addition to deep well pumping equipment. Under certain conditions, it is probably the most efficient device for lifting water from deep wells where relatively high rates of pumping are required.

PUMP COMBINATIONS

It is often desirable or necessary to use booster pumps where water is to be delivered from a considerable depth in a well against a high pressure at the ground level. The turbine pump or the bucket pump can be used to pump against considerable pressure heads at the surface while the air lift pump is seriously limited in this respect. Because of the necessity of transmitting all energy for pumping through long rods or shafts in the case of bucket and turbine pumps to relatively inaccessible parts, it is advisable in most cases to relieve the deep well pump of the pressure head at ground level. By taking over a considerable portion of the pumping load on a booster pump, greater overall pumping efficiency, less pump maintenance cost, and greater reliability will usually be secured. Booster pumps may be connected directly to deep well pumps or a reservoir may be used between the two pumps. Where sand is carried in water, a reservoir is very essential for the removal of the sand so that it will not be delivered into the water mains. It is difficult to obtain pumps with characteristics that will result in a correct division of load between the deep well pump and the booster pump, when they are directly connected, with varying conditions in the well and with variable pressure heads in the water distribution system. A reservoir permits the pumps to operate at different rates and this reduces the influence of variable pumping conditions.

The efficiency of a unit consisting of a deep well pump and a booster pump is:

$$E = \frac{H_D + H_B}{\frac{H_D}{E_D} + \frac{H_B}{E_B}}$$

where H_D = Head delivered by deep well pump.
H_B = Head delivered by booster pump.
E_D = Efficiency of deep well pump.
E_B = Efficiency of booster pump.

The efficiency of the deep well pump, of course, has less influence on overall economy as the percentage of the head delivered by that pump decreases. If, for example, one-tenth of the total head is furnished by the deep well pump and nine-tenths by the booster, with E_D = 30 percent and E_B = 60 percent, the resulting overall efficiency would be 54.5 percent.

THE RECIPROCATING OR BUCKET PUMP

The mechanism of the ordinary power driven deep well bucket pump is simple and therefore requires only ordinary mechanical skill in its maintenance. Because of the necessity of transmitting energy through very long reciprocating rods, the speed of operation is necessarily slow so that deterioration of bearing in the pump head is due primarily to heavy loads or impact rather than to high rotative speeds. Because of intermittent action, the water in the long discharge pipe must be accelerated from zero velocity to some maximum value once or twice per revolution of the crank shaft in the ordinary single and double acting pump respectively. With a triple acting pump, the flow is more steady, and since the velocity of flow in the pipe does not reach zero during a cycle of operation, the accelerating forces required are less and therefore, the strains in the pump rods and impacts on the driving mechanism are less than in the single or double acting pumps when operated at the same speed. The stresses in the pump rods and in the driving head caused by forces required to accelerate the water and the reciprocating parts of the pump vary with the square of the speed of operation. In order to permit higher speeds of operation or to reduce impacts at a given speed, the motion of the simple crank and connecting rod has been modified by certain manufacturers in recent years. The special

feature of the newer type of pump head mechanisms is the reduction of the ratio of maximum velocity to average velocity of the buckets during the working stroke. This is accomplished by using more than 180° of rotation of the main crank shaft during the working stroke thus necessitating a comparatively quick return of the bucket on the down stroke. These motions permit an overlapping motion of buckets on the working stroke so that the column of water in the dis-

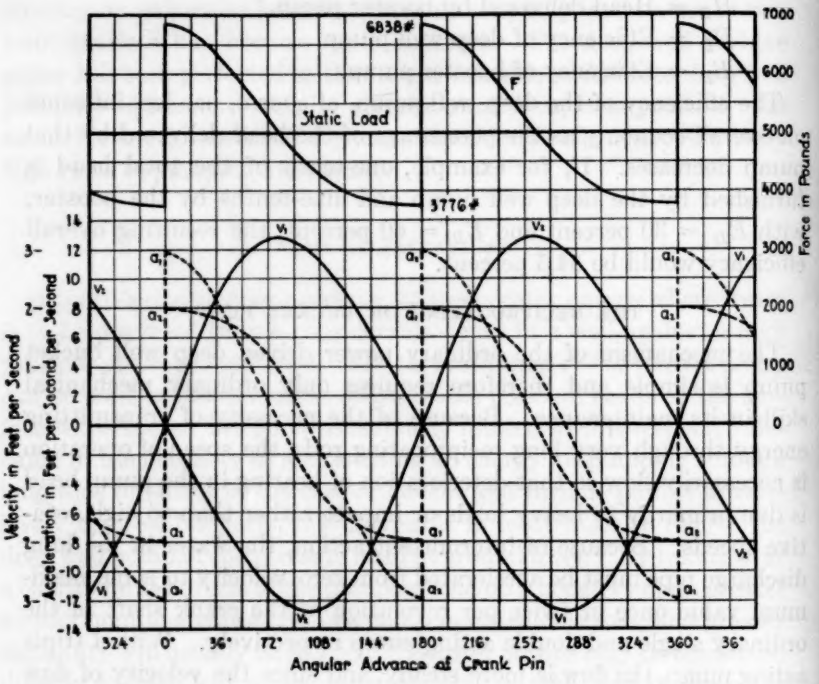


FIG. 1

charge pipe is kept in motion at all times even with two single acting buckets.

In figure 1 are shown the calculated velocities and accelerations of buckets, and pump rod loads at all points in a complete cycle of operation of a simple duplex pump with a 24-inch stroke running at 30 r.p.m. Forces required to accelerate the pump rods have been neglected as well as frictional resistances encountered in the pipe. The tension in each pump rod varies from 3860 to 6592 pounds with

a static load or with a pressure equal to the head pumped against of 5000 pounds. The three sets of curves in figures 2, 3, and 4[4] show the same quantities for a Johnson, a Chippewa, and a Luitwieler pump for approximately the same lengths of stroke and speed, and each with two single acting buckets. The designs assumed for these calculations were scaled in some cases from manufacturer's diagrams because detailed design features were not available. Maxi-

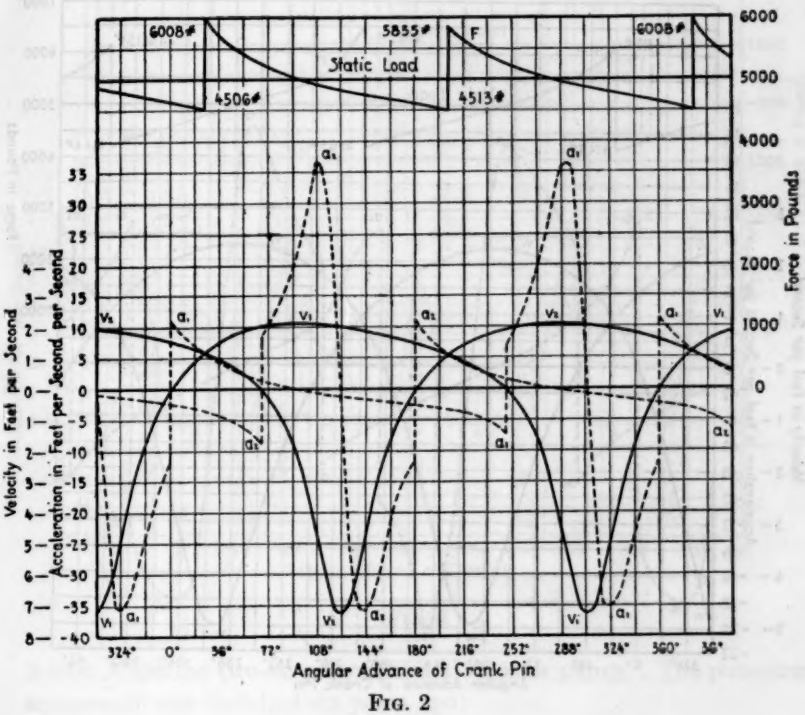


FIG. 2

mum and minimum pump rod loads were 6008 and 4506 pounds for the Johnson pump, and 6525 and 4144 pounds for the Chippewa pump. The cams of the Luitwieler pump are of such shape that the idle bucket is accelerated up to a velocity equal to that of the working bucket before the load comes into it. A uniform-rate of pumping is thus secured and no forces are required for accelerating the column

[4] Taken from Advance Sheets on "Hydraulic Machinery" by Prof. Daniel W. Mead.

of water. Of course, forces are involved in accelerating the pump rods, but these forces have not been shown in any of the illustrations. In some of these pumps there has been an appreciable equalization of velocity of flow of water (in one case complete equalization) and a reduction of load changes on pump rods at the end of strokes. These improvements have been secured in some cases at a considerable sacrifice in simplicity and ruggedness of the driving head.

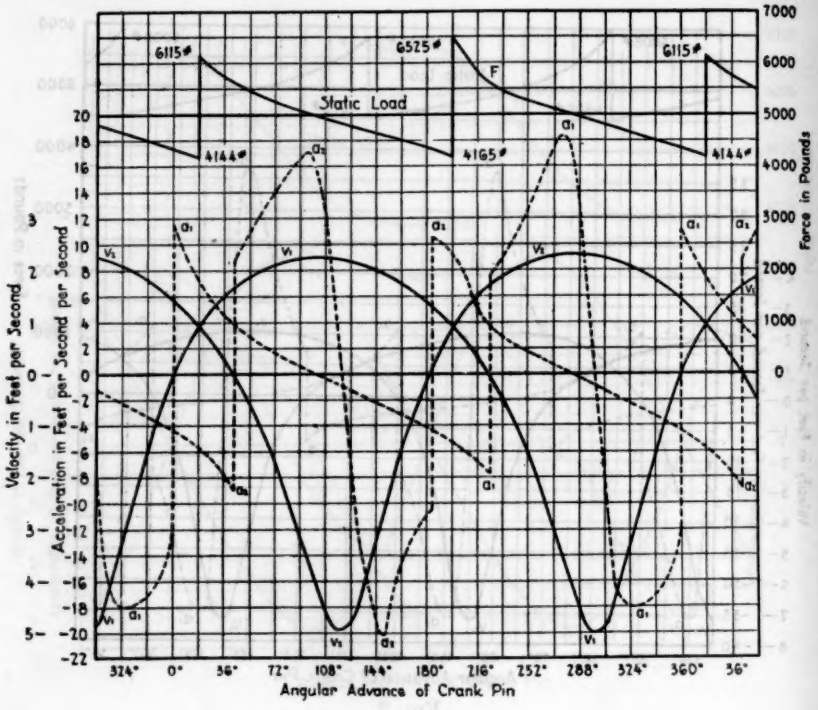


FIG. 3

The rates of pumping possible with the ordinary type of driving head are relatively small for deep settings of bucket pumps when compared with that which may be secured from a given well with other deep well pumps. For example, one manufacturer of reciprocating deep well pumps indicates 490 gallons per minute as the maximum rate of pumpage attainable with a double stroke geared pump from a 12-inch well. Probably one of the greatest disadvantages of this type of pump is its relative low capacity.

The bucket pump is used primarily in private wells or in small

water supply systems. For such uses reliability and simplicity of the pump are usually more important than high efficiency. Maintenance of this type of pump does not usually require highly skilled mechanics.

Following are the results of tests recently made by the writer on a deep well pumping unit consisting of a Chippewa triple stroke pump which delivered into a small reservoir near the well, and a 5-inch by

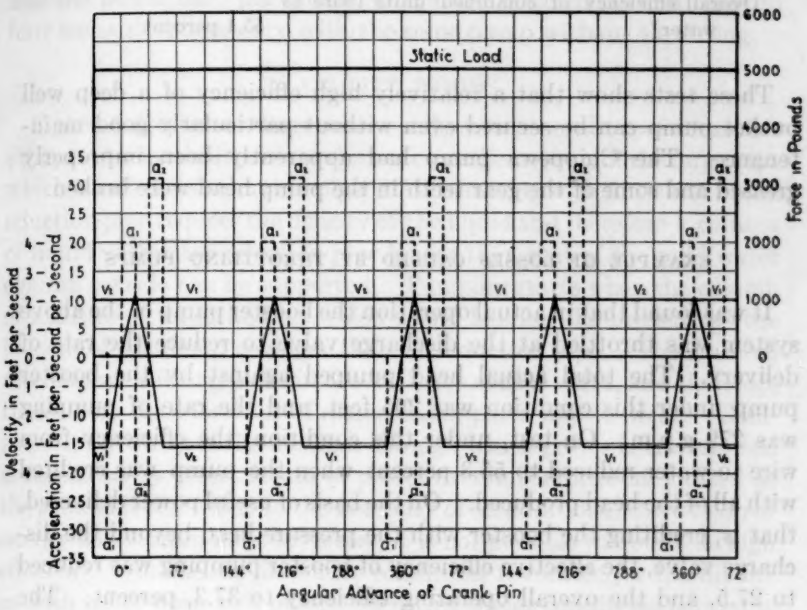


FIG. 4

5-inch Manistee two-stage contrifugal booster pump. The pumping equipment was installed six years ago:

Chippewa pump

Diameter of barrel, 11½-inch stroke	36 inches
Depth of barrel	155 feet
Revolutions per minute	14.45
Rate of pumping	318.5 g.p.m.
Total pumping head	173.0 feet
Power output of pump	13.92 horse power
Electrical input to motor	19.1 kw. = 25.6 h.p.
Efficiency of unit (wire to water)	54.5 percent
Pump efficiency (allowing 85 percent motor and belt efficiency)	64 percent

Booster pump

Rate of pumping	626 g.p.m.
Total head	190.5 feet
Power output of pump	30.0 horse power
Power input to motor	37.7 kw. = 50.5 h.p.
Efficiency of pump and motor	59.5 percent
Pump efficiency (allow 85 percent motor efficiency)	70.0 percent
Overall efficiency of combined units (wire to water)	55.1 percent

These tests show that a relatively high efficiency of a deep well bucket pump can be secured even without particularly good maintenance. The Chippewa pump had apparently been improperly greased and some of the gear teeth in the pump head were broken.

EXAMPLE OF LOSSES CAUSED BY THROTTLING PUMPS

It was found that in actual operation the booster pump of the above system was throttled at the discharge valve to reduce the rate of delivery. The total actual head pumped against by the booster pump under this condition was 295 feet, and the rate of pumping was 279 g.p.m. On test, under this condition, the efficiency from wire to water reduced to 55.3 percent when the pump was credited with all of the head produced. On the basis of useful power delivered, that is, crediting the booster with the pressure head beyond the discharge valve, the effective efficiency of booster pumping was reduced to 27.5, and the overall operating efficiency to 37.3, percent. The kilowatt hours required per million gallon without throttling would be 2,000, while the energy actually used with a throttled discharge valve would increase to 2735 or to about 37 percent more than it should have been with this equipment. It was found at this well that the water meter over-registered the quantity of water pumped by 63 percent. The electrical consumption for the year 1929 did not appear excessive on the basis of meter registration of pumpage. The average consumption of energy for the year 1929 was 2810 k.w.h. per million gallons with water meter registration corrected on basis of these tests. Since about 150 million gallons were pumped in a year, the energy wasted by throttling the booster pump was 810 by 150 or 121,500 k.w.h. which at $1\frac{1}{4}$ cents per kilowatt hour represents a loss of $1520 per year.

The writer has visited four small public water works plants during the past year in which the rate of pumping with centrifugal pumps is normally reduced by throttling the discharge valve. The losses involved are due not only to the additional head against which all water must be pumped, but also to the fact that the pump itself is operated at a poor efficiency point. In one plant a 1000 gallon per minute pump delivers at an average rate of 300 gallons per minute, and the power used per quantity of water pumped is from three to four times that necessary with the same pump without throttling.

AIR LIFT PUMPS

Figure 5 illustrates diagrammatically two general types of air lift pumps. Air is admitted into the water at the bottom of the large pipe which is called the eduction pipe. Air in mixing with the water in the eduction pipe reduces the density of the liquid and therefore a column of liquid within the eduction pipe longer than that of the solid water column outside can be supported. Pumping starts when the column of mixture of water and air which would balance the pressure at the bottom of the pump is longer than the eduction pipe. There are no mechanical working parts in the well, not even a valve, and therefore there is no deterioration of inaccessible parts due to wear. Compressed air which is the medium through which power is transmitted is furnished by direct steam or power driven compressors located at the ground level. Due to the necessity of an additional conversion of energy and certain inherent losses, it is not possible to obtain as high efficiency with the air lift under conditions favorable for the air lift as with other deep well pumps operating under conditions favorable for these pumps. There are many conditions, however, under which the air lift pump will give operating efficiencies as high as are obtainable with other pumps. The simplicity, reliability, freedom from breakdown, and the accessibility of moving parts in the air lift pump may justify its use although its actual operating efficiency is less than that of other pumps under certain conditions.

There are many air lift installations which are uneconomical to operate because of faulty design of the pump or improper operation. It is so simple to place a pipe in a well and to deliver air at the lower end of the pipe that practically any one can install an air lift pump. Many pumps are used which are entirely inadequate. Because of small well casings and poor alignment of drilled holes, air lift pumps have sometimes been used because no other type of pump could be

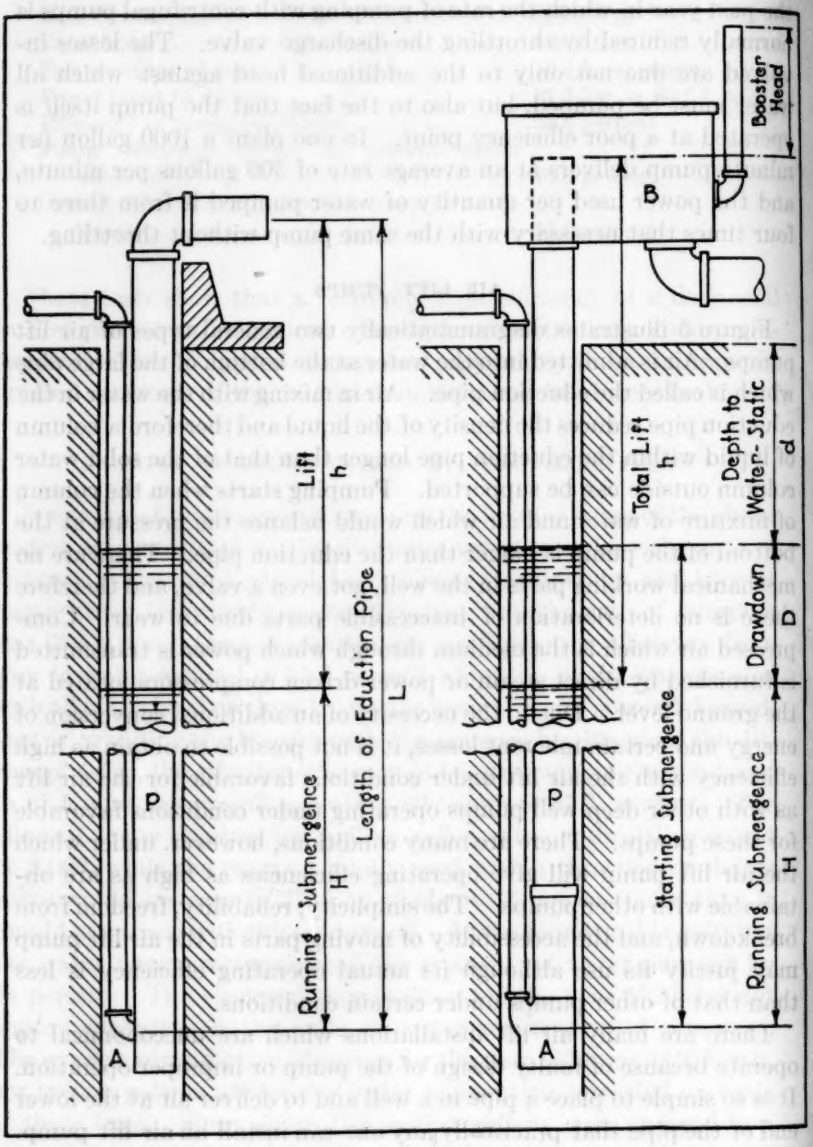

Booster Head
B
Lift h
Length of Eduction Pipe L
Running Submergence H
P
A
Total Lift h
Depth to Water-Static d
Drawdown D
Starting Submergence
Running Submergence H
P
A

FIG. 5

placed in such wells. The eduction pipes which can be installed under these conditions are often much too small for the rates of pumping desired, and pumps are usually run at rates which are entirely too great for economical operation. It is possible to double the air input, that is, to double the power input, without increasing the rate of pumping at all. The writer has seen the rate of pumping increase with reduction of the air supply both in laboratory and in field tests. Improper depths of submergence and unnecessary resistances to flow at the top of the eduction pipe result in losses which may seriously influence operating costs. In some installations the air and water are carried through long horizontal pipes which are connected directly to eduction pipes.

Air lift pumps are usually designed on the basis of the formula

$$V = \frac{h}{C \log \frac{H + 34}{34}}$$

where h = the lift.
H = the submergence of foot piece.
V = free air required per gallon of water.
C = coefficient which varies only with the efficiency of the pump

The formula is derived by equating the work done in lifting one gallon of water (h) feet to the energy required in the air at the foot piece, assuming isothermal expansion of the air in the eduction pipe. After determining the air requirements, the designer uses his judgment as to sizes of pipe and velocities of flow, and hopes that the efficiency assumed in the above formula will be realized. A more scientific method of design recognizes individual factors such as the losses involved with varying velocities, different mixtures of air and water, length of eduction pipe, entrance losses and the velocity head at point of discharge. The rate of pumping which can be secured with any given rate of air supply can be computed by the latter method.

The frictional resistances in the eduction pipe vary with the velocity of flow, the ratio of air to water by volume, and the size of eduction pipe. Figure 6 shows the frictional losses in 3-, 6-, and 8-inch eduction pipes. The friction losses vary from 3 to 7 times that which would be experienced with water alone flowing at the same velocity in particular pipes for ranges of velocities commonly used.

With data such as that presented in figure 6, the losses due to flow in the eduction pipe with a particular ratio of mixture of air and water

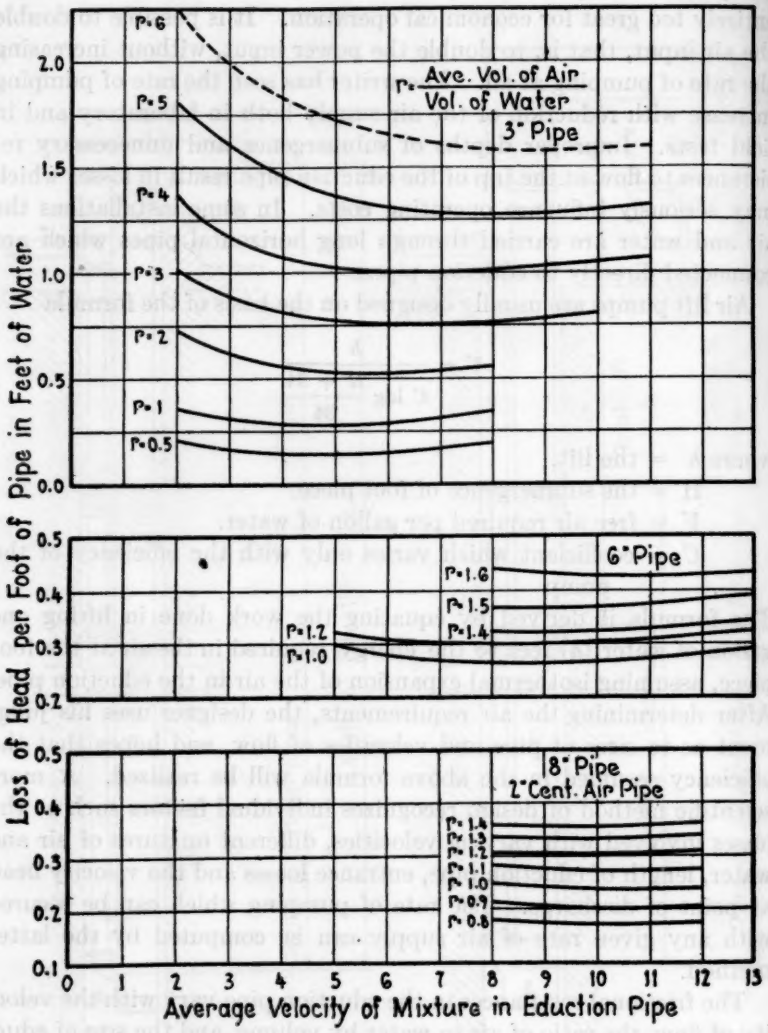


FIG. 6

can be determined. The energy in the air at the foot piece must equal that required for the useful work plus all losses. Since the losses vary with the velocity and with the ratio of air to water which

in turn depends upon the rate of supply of air, a design consists of determining the above balance by a trial and error method. A

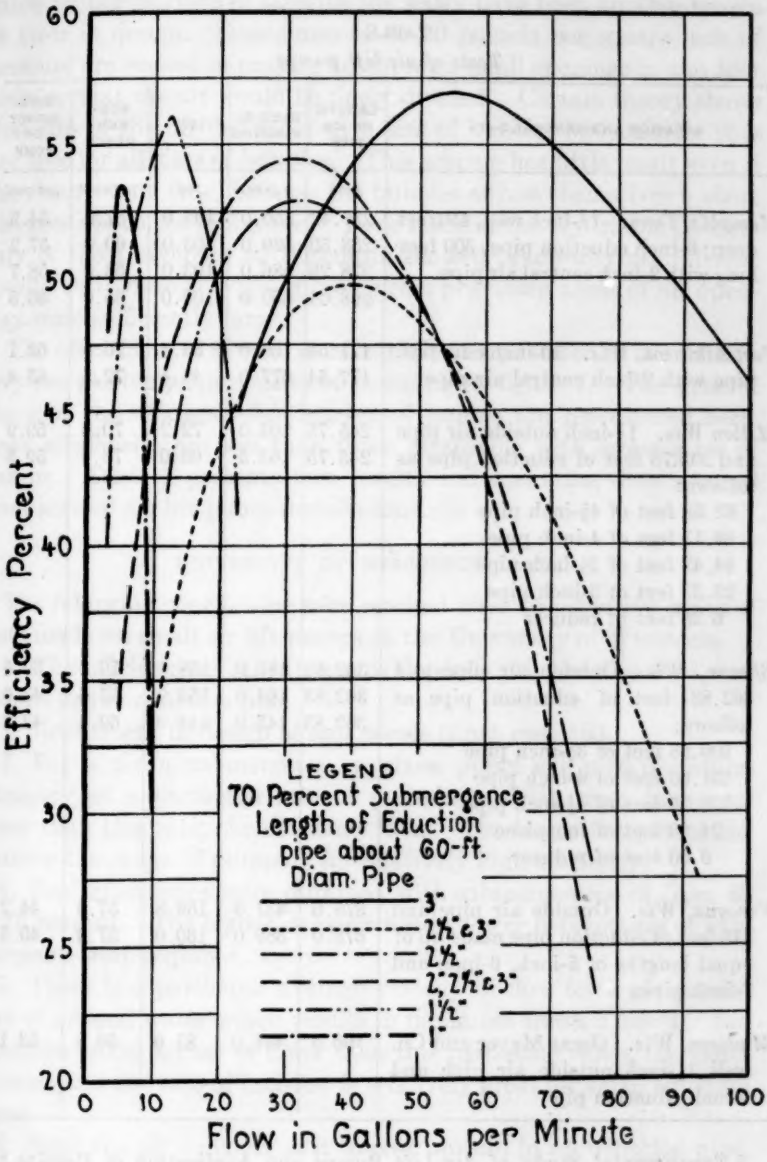


FIG. 7

further discussion of air lift pump design is presented in a University of Wisconsin Bulletin.[5]

TABLE 1

Tests of air lift pumps

LOCATION AND GENERAL DATA	LENGTH OF AIR PIPE	RATE OF PUMPING	LIFT	SUBMERGENCE	EFFICIENCY OF PUMP
	feet	*g.p.m.*	*feet*	*per cent*	*per cent*
Memphis, Tenn. 14-inch well, 450 feet deep; 8-inch eduction pipe, 300 feet long with 2-inch central air pipe	237.42	550.0	102.0	57.0	54.3
	258.50	520.0	103.0	60.0	57.2
	278.29	586.0	103.0	63.0	58.7
	298.04	650.0	105.0	65.0	66.5
Fort Atkinson, Wis. 10-inch eduction pipe with 2-inch central air pipe	177.54	766.0	53.3	70.0	62.1
	177.54	677.0	48.4	72.8	65.4
Milton Wis. 1¼-inch outside air pipe and 245.75 feet of eduction pipe as follows: 62.58 feet of 4½-inch pipe 69.17 feet of 4-inch pipe 84.42 feet of 3½-inch pipe 23.33 feet of 3-inch pipe 6.25 feet of reducer	245.75	203.0	72.2	70.6	59.9
	245.75	163.5	66.0	73.1	59.8
Sharon, Wis. Outside air pipe and 362.83 feet of eduction pipe as follows: 109.58 feet of 3½-inch pipe 101.00 feet of 4-inch pipe 122.25 feet of 4½-inch pipe 24.00 feet of footpiece 6.00 feet of reducer	362.83	181.0	158.1	56.3	39.6
	362.83	164.0	153.9	57.6	43.0
	362.83	145.0	144.0	60.4	42.2
Viroqua, Wis. Outside air pipe and 375 feet of eduction pipe made up of equal lengths of 5-inch, 6-inch and 7-inch pipes	375.0	437.0	159.8	57.4	44.2
	375.0	389.0	160.0	57.4	45.5
Madison, Wis. Oscar Mayer and Co. well 1½-inch outside air pipe and 6-inch eduction pipe	196.5	401.0	81.0	58.8	53.1

[5] Experimental Study of Air Lift Pumps and Application of Results to Design, Vol. IX, No. 4.

Certain faulty basic assumptions, such as the need for finely divided air bubbles and certain peculiar passageways in foot pieces to cause proper mixture of air with the water have been another source of error in design. Sometimes 15 or 20 pounds per square inch of pressure are wasted in passing air through small openings in the foot piece so that the air would be finely divided. Certain theory shows that slip of the air decreases with size of bubble, but actually it is not true for all sizes of bubbles. This scheme has little merit even if the theory were true, because the bubbles adjust themselves a short distance above the foot piece. Tests made at the Hydraulic Laboratory of the University of Wisconsin showed practically no different results with various types of foot pieces providing areas of air openings were sufficiently large.

The efficiencies of several small air lift pumps tested in the laboratory are plotted against rates of pumping in figure 7. The general shape of the efficiency-flow curves of pumps are represented here. Peak efficiencies are less than those attainable with larger sizes of pumps. Table 1 presents test results and operating data secured from several air lift pump installations.

UNIVERSITY OF WISCONSIN TESTS

The following conclusions were reached after an extensive series of tests made on small air lift pumps at the University of Wisconsin:

1. The efficiency of air lift pumps depends primarily on flow conditions in the eduction pipe.

2. Refinement in design of foot pieces is not essential.

3. For a given submergence, a given pump will give maximum efficiency at a particular rate of pumping. With rates higher or lower than this rate, the efficiency falls. The larger the pump, the broader the range of pumping for relatively high efficiency.

4. Best efficiencies were obtained with submergencies of from 65 to 75 percent. For higher delivery heads, lower percentages of submergence were required.

5. There is a particular average velocity of flow for a given mixture of air and water which results in minimum friction losses. For velocities either higher or lower than this particular value, the losses increase and the rate of increase is relatively great for small eduction pipes.

6. Since the air expands as it moves upward in an eduction pipe, velocities of flow increase. The use of larger sizes of pipes in the

upper portion of pumps results in lower velocities. Tests show that higher rates of pumping can be secured in the high efficiency range by enlarging the upper end of the eduction pipe. Peak efficiencies were not increased over those obtainable with eduction pipes of constant diameter.

AIR LIFT PUMP INSTALLATIONS, TESTS, AND OPERATING RESULTS

Madison Unit, Well No. 3

This unit well plant was installed in March, 1929, and consists of the following:

1—754-foot well 20 to 15¼ inches diameter.
1—Central air pump. Eduction pipe 12 inches in diameter, 375 feet long. Air pipe 3 inches in diameter, 331 feet long.
1—Worthington horizontal duplex air compressor 15 inches by 9½ inches by 12 inch stroke, direct connected to a General Electric synchronous motor.
1—30,000 gallon reservoir.
1—Allis-Chalmers 8 inches—6 inches 1600 g.p.m. centrifugal booster pump, directly connected to an Allis-Chalmers induction motor.

Test of air lift pump

Rate of pumping	1730 g.p.m.
Delivery head	116.9 feet
Submergence	214.1 feet = 64.7 percent
Power output of pump	51.1 horse power
Isothermal energy in air at foot piece	69.5 horse power
Efficiency of pump $\frac{\text{Water H.P.}}{\text{Isothermal Energy}}$	73.5 percent
Input to motor	128.1 horse power
Efficiency, wire to water	39.9 percent
Ratio air to water in eduction pipe by volume	0.776
Average velocity of mixture	9.21 ft./sec.
Loss of head per foot of eduction pipe	0.129 feet

The loss with water alone flowing at a velocity of 9.21 is estimated as 0.082 foot per foot of pipe, so that the loss of head with the mixture was about 58 percent greater than with water alone.

Centrifugal booster pump test

Head	211.5 feet
Rate of pumping	1415 gal./min.
Wire to water efficiency	80.6 percent

On the basis of this test the overall efficiency of the two pumps (wire to water) is 59.2 percent.

During the month of August, 1929, 42.7 million gallons were pumped against a total average head from the well to the mains of 302.5 feet. The power consumed was 71,230 k.w.h. which gave an operating efficiency of 57 percent from wire to water. At 1.25 cents per kilowatt hour, the energy cost of pumping at this well would be $20.85 per million gallons.

MADISON'S CENTRAL AIR LIFT SYSTEM

Near the central water works pumping plant in Madison there are ten air lift pumps which are supplied air by steam driven compressors located at the pumping station. Water from the wells is returned to a reservoir at the central plant for which it is pumped into the mains by high duty steam pumps. In a test conducted in 1919 on five of the wells, the pumps tested had eduction pipes varying from 5 to 7 inches. The minimum rate of pumping from one well was 292 g.p.m., the maximum was 920 g.p.m., and the average rate per well was 466 g.p.m. In this test one Ingersoll Rand 14/23 twenty-inch cross compound piston valve air compressor was used.

Duration of test	3 hours
Average total pumping head	84.2 feet
Total water pumped	418,958 gallons
Speed of compressor	90 r.p.m.
Steam pressure	150 lbs. per sq. in.
Air pressure	90 lbs.
Steam used	5956 pounds
Useful work done	294,607,028 ft. lbs.
Duty	49,480,000 ft. lbs./1000 pounds of steam

This duty corresponds to a steam rate of 40.22 pounds of steam per delivered water horse power hour.

A 2500 cubic foot horizontal two-stage cross-compound Nordberg Corliss air compressor installed in the Madison central plant in 1924 showed the following test results:

Air pressure	101 pounds per sq. in.
Free air delivered (117 r.p.m.)	2508 cu. ft. per min.
Indicated steam power	454.31 horse power
Indicated air power	442.00 horse power
Steam pressure	150 lbs. per sq. in.
Steam rate	12.08 lbs. per steam I.H.P.
Steam rate	12.43 lbs. per air I.H.P.
Power to compress air isothermally	298.5 horse power
Efficiency, isothermal to steam I.H.P.	65.7 percent

On the basis of 60 percent efficiency of an air lift pump with air delivered by the above compressor, and assuming a 10-pound per square inch loss in pressure due to transmission and regulation of air flow, the following results would be secured:

	percent
Efficiency of air transmission	95.2
Efficiency of pump	60.0
Efficiency of compressor (isothermal to steam I.H.P.)	65.7
Overall efficiency from I.H.P. in steam cylinder to water	37.5

One water horse power requires 2.67 I.H.P. in steam end of compressor. The steam required per water horse power would be 32.2 pounds, corresponding to a duty of 61.6 million foot pounds per 1000 pounds of steam.

DUBUQUE, IOWA, AIR LIFT INSTALLATION

In September, 1924, tests were run on an air lift installation at Dubuque, consisting of five 8-inch (2½-inch central air pipe) air lift pumps and a Sullivan 20-inch by 12-inch by 14-inch twin angle compound two-stage belt driven compressors. Rate of pumping was 4497 gallons per minute with an average lift of 128 feet and a submergence of 65 percent. The free air used per gallon was 0.442 cubic feet. Power delivered to the motor was 412 horse power. The wire to water efficiency was 35.4 percent, and the pump efficiency was 58 percent.

ADDITIONAL TESTS OF AIR LIFT PUMPS

A summary of results of tests obtained on several air lift pumps is given in table 1. Data were not sufficiently complete to obtain the overall efficiencies of these installations so that pump efficiencies only are presented.

The test of the Memphis unit was made on an air lift pump of the water supply system at the time engineers for that city were considering a new system of wells and pumps. After a careful study of various types of deep well pumps, air lift pumps were selected for Memphis.[6]

A normal supply of 18,000,000 gallons per day was to be pumped from wells, the estimated lift being 72 feet. Analyses showed the first cost of the air lift system was less than that of a deep well elec-

[6] Report on Memphis Water Supply, City of Memphis. Fuller and McClintock.

trically driven turbine pump installation of equal normal and reserve capacity, the saving being about 7.5 percent. Current was to be generated by 500 k.w. turbo generators for the turbine pump system. Total operating and fixed costs based upon efficiencies determined from operating records of existing plants showed an estimated saving of $9360 per year or a saving of about 7.8 percent with the air lift over that with the alternate type of deep well pump.

DEEP WELL TURBINE PUMPS

The deep well turbine pump is used for lifting water from wells to the ground surface or for pumping directly into water mains against considerable pressures. A well installation may consist of a turbine pump and a high pressure booster pump at ground level with a reservoir intervening between the pump or with direct connection of the two pumps. One type of turbine pump has centrifugal booster rotors attached to the turbine shaft at or near ground level.

In recent years, the efficiency of turbine pumps has been materially improved and the range of sizes and capacities has been increased. Pumps have been developed for use in wells as small as four inches, and for large wells with capacities up to 4000 gallons per minute and heads as high as 640 feet. Improvements in bearings, in the lubrication and protection of submerged bearings, and in the hydraulic design of the pump, make it easier to secure higher sustained operating economies than was possible with older pumps of this type. It is very important in the selection of a pump of this kind for a particular installation to know the characteristics of the well in which it is to be used because the rate of pumping and the resulting efficiency are so intimately related to the head to be pumped against.

The exent to which sand may be carried in the water and corrosive tendencies of the water should be investigated because these factors may seriously influence the performance of turbine pumps. Where sand is encountered it is essential to remove it before pumping the water into the high pressure system. The presence of sand, of course, precludes direct pumping from wells into the water mains. With some of the older types of turbine pumps in Madison, where some sand is encountered, a decrease of 3 to 5 percent per year in the efficiency was experienced. At Lincoln, Nebraska, it is reported that the effect of sand is such that the average life of a turbine pump is about five years and that it is necessary to pull pumps two or three times per year for repairs.

TABLE 2

Deep well turbine pumps, tests, operating economics

INSTALLATION	DATE INSTALLED	DATE OR PERIOD OF TEST	RATE OF PUMPING	TOTAL AMOUNT PUMPED	HEAD DELIVERED		EFFICIENCY—PER CENT WIRE TO WATER		
					Turbine	Booster	Turbine	Booster	Combination
			g.p.m.	*million gallons*	*feet*	*feet*			
Madison, Wis.—Unit Well No. 1. Aurora pump combination with booster in one unit	May, 1924	Feb., 1925	1,233		280				59.0
		Year 1927		225.4	283 Ave.				54.5
		Year 1929		433.5	286 Ave.				53.6
Madison, Wis.—Unit Well No. 2. Layne Bowler Turbine Pump Delivering direct to mains	Sept., 1924	Feb., 1925	2,620		389		54.0		54.0
		Year 1928		153.6	389		50.6		50.6
Madison, Wis.—Unit Well No. 2. Peerless deep well turbine pump replaced above pump	1929	Aug., 1929	2,288		398		57.0		57.0
		Mo. of Sept., 1929		18.3	395		59.8		59.8
*Wauwatosa, Wis.—Unit Well No. 4. Layne turbine pump, direct Conn. to Allis Chalmers centrifugal booster	1927	Mar., 1930	894		364	144.7			61.5
		Year of 1929		352.6					58.9
Wauwatosa, Wis.—Unit Well No. 5. Layne turbine pump delivering directly into mains		Mar., 1930	721		458		54.2		54.2

—— Wis.—Unit Well No. 3. Layne pump delivering to Manistee centrifugal booster	1929	Mar., 1930	259		244	91.5			48.2
†Viroqua, Wis. Deep well turbine pump, reservoir, and centrifugal booster		Year of 1929		63.0	137	93.0			26.5
Burlington, Wis. Deep well turbine pump, reservoir and centrifugal booster		Mo. of Feb., 1930		7.9	110	157.0			48.5

* Reported as the deepest setting of a turbine pump in Wisconsin. Depth of bowls 380 feet.

† A test of this same well with an air lift pump, motor driven compressor, and the present booster equipment was made in 1924. The wire to water efficiency at that time was found to be 43 percent.

A turbine pump at Winona, Minnesota, was removed lately from a well of the water supply system for that city. Due probably to some corrosive acton of the water or to electrolysis, the pump bowls, the impeller, guide vanes, bearings, and the shaft had seriously deteriorated. There were holes extending through the walls of the bowls; impeller blades were honey-combed so that edges of the metal could be broken off easily; and the pump shaft was so corroded that key seats in the shaft were just distinguishable. Guide bearings were cut so that the inner diameter of the bearing was about ⅛-inch larger than the shaft. This pump had been in the well for twenty-five months. The capacity of the pump prior to its removal had been reduced to about 60 percent of its original capacity.

Operating and test results secured for several turbine well installations are shown in table 2. The first five units are in systems in which maintenance is of the highest order. Gages and meters are available and are read regularly, and the performance of each unit is closely watched.

Table 2 shows very excellent test efficiencies and operating economies for some installations. It is reported that there has been no expenditure for repairs on the turbine pump in Well No. 4 at Wauwatosa, although it has been in operation about three years. The average wire to water efficiency of 59.8 percent attained for the year 1929 is noteworthy. The turbine pump in Well No. 1 of Madison gives very satisfactory operating economies, and this pump has been pulled but once for maintenance operations. Experience in Well No. 2 at Madison was not as satisfactory. The first turbine pump was replaced after about four years. At the time of removal the efficiency of the pump was less than 50 percent, and its capacity had decreased materially. Lack of knowledge of the characteristics of Well No. 3 at ————, Wisconsin, at the time that the turbine pump was selected, may account for the lower efficiency experienced there. At the time the writer visited the Viroqua plant, the thrust bearing of the turbine pump was giving trouble, and there was a considerable amount of metal in the lubricating oil. This bearing had been replaced a short time before this inspection. Probably mechanical troubles in the turbine pump are responsible for the poor showing of that plant.

If a group of wells equipped with motor driven pumps were supplied energy by a 500 kilowatt turbo generator set, a test duty of from 60 to 70 millions foot pounds per 1000 pounds of steam could be secured.

CONCLUSIONS

The reciprocating bucket pump gives very good efficiency and works satisfactorily in many wells where relatively small rates of pumping are required. Difficulty of maintenance when sand is encountered, where pump rods cannot be suspended straight and in a vertical position, and where high rates of delivery with high lifts are required, are usually such as to make some other pump desirable. The most satisfactory use of this pump is in private systems.

A system of wells equipped with air lift pumps and with air supplied by modern efficient steam-driven air compressors located at a central plant, gives duties which are favorable to this system. Other advantages of the air lift are: low first cost, low maintenance cost, accessibility of working parts, and reliability. Aeration of the water is claimed as an advantage in some installations. The air lift pump can be used satisfactorily in unit well installations.

A disadvantage of the air lift is the need for a considerable submergence of the eduction pipe. In some shallow wells the use of this type of pump is not practicable. When improperly designed or where conditions in the well will not permit the correct installation, it is apt to be very inefficient, and very costly to operate.

Where the major part of the pumping head is delivered by a booster pump at a unit well, the overall efficiency may be quite high even with a relatively poor air lift pump installation.

The turbine deep well pump properly selected and installed will give very good efficiency. Where pressures at the ground level are not too high, this pump can often be used to pump directly into the mains. This type of pump with automatic control requires very little attendance. In this regard it has the advantage over a unit well installation with an automatically controlled electric motor driven air compressor and an air lift pump.

Where electrical energy is purchased at a high unit cost, the isolated unit with a turbine pump has an advantage if high efficiencies can be maintained, even though costly maintenance is required to secure the high efficiency.

The disadvantage of this pump is the short life and high maintenance cost when used in certain wells. Deterioration of the pump is sometimes so rapid that its actual average operating efficiency is low. For equal reliability and continuity of service, more wells and pumps are required with turbine pumps than with air lift pumps.

DISCUSSION

L. A. SMITH:[7] I want to commend Mr. Ward for his clear cut discussion of the merits of various types of deep well pumps. The figures he submits of actual operating results are of great value to engineers with deep well pumping problems. I agree heartily with his statement that "no particular type of deep well pump has yet been developed to meet all contingencies encountered so that it can be recommended in exclusion of all other types for all requirements."

Having had experience with only two types of deep well pumping, namely, air lift and turbine pumps, I will confine my brief discussion to these two methods. The highest operating efficiency, 61.5 percent, quoted by Mr. Ward was obtained from the turbine pump at Wauwatosa. The efficiency of the combination air lift and centrifugal pump at Madison is 59.2 percent. In addition, the cost of attendance at Madison is somewhat greater, but will be reduced when the complete automatic equipment is completed.

In spite of the difference in efficiency and operating costs I believe that the Madison installation is based on sound engineering principles, under the local conditions, and I want to outline briefly the reasons that led us to use the air lift combination rather than the deep well turbine.

1. Our wells produce a considerable quantity of sand which caused efficiencies of deep well pumps to drop off from 1 to 3 percent per year. The horizontal booster does not drop off materially in efficiency because clear water only is pumped, due to the fact that the sand is removed in a settling basin. The pumping of even a small amount of sand into the distribution system cannot be tolerated.

2. Aeration of the water by air lift pumping effectively removes the iron from the water in Madison so that red water does not occur in the distribution system. The amount saved in Madison in the cost of pumping water for flushing is over $1000 annually as flushing has been reduced nearly 80 percent.

3. In air lift deep well pumping the moving units are always open to inspection, while the turbine pump is located in the well. This materially reduces the likelihood of an enforced shut-down due to failure, and if a failure occurs the time necessary to make repairs to a compressor is generally a matter of hours, rather than days as it is necessary to pull the pump for repairs. It is, therefore, necessary to

[7] Superintendent Waterworks, Madison, Wis.

provide an additional unit for equal reliability which materially increases fixed charges.

While I recognize that high efficiencies and low operating costs are desirable attributes of pumping equipment it is unsound to attempt to obtain them if sand or iron troubles are increased or reliability is adversely affected.

A modern water utility is a success that furnishes an ample quantity of pure water at all times under proper pressure, and it is a failure if it fails in this type of service even if the rates are lower. This is not a justification of the water rates in Madison, because they are the third lowest in the state of Wisconsin and are less than half of the average rate in Wisconsin cities over 10,000 in population. In spite of this rate level, the water utility in Madison makes a net profit of over $2 per capita annually after paying operating expenses, taxes and setting aside an adequate reserve for depreciation.

D. F. BIXLER:[8] Mr. Ward's paper presents a good, impartial picture of the different types of well pumps, and will be greatly appreciated by those who have to pick types for certain installations. So many problems are involved in this selection, that it is frequently a difficult proposition.

The outstanding advantage of the air lift pump is the fact there are no moving parts in the well. The only machine is a compressor, which is in the station under the care of an attendant, and this makes it a very reliable installation. It also means that the only piece of equipment which will have to be provided to give good returns of service is a compressor. It will not be necessary to provide an extra well as in the case of other pumps. This means that the cost of the air lift installation is less than the other types, unless the water bearing soil is underlaid with hard rock in which the air lift well must be driven.

The outstanding advantage of the turbine type pump is its adaptability. It can be operated either as a low lift pump or can be fitted with a booster pump and operate directly into the lines. It starts and stops automatically and does not require an attendant. In a small community, such a small pumping unit can form the total pumping plant. As the community grows, additional units may be added as required. This feature is also advantageous in that, by distributing the complete and independent units throughout the dis-

[8] American Water Works and Electric Company, New York, N. Y.

tribution system, large investments in mains may be eliminated. In a fairly good sized plant, such units may be used to good advantage and in an extreme case, such units might take care of the peak load with an advantage that it will eliminate additional capacity in low lift pumps, boilers, and even distribution mains.

The turbine pump probably has a little greater efficiency than the air lift pump, but this is partially offset by the greater first cost. Sand in a well will wear the impeller veins and reduce the efficiency, but if the well is properly cleaned before the pump is installed, this should proceed very slowly, and if the impeller and wearing rings are renewed after five years or so, the average efficiency may be maintained quite close to the efficiency originally obtained.

An air lift installation has no wear in the wells as it has no moving parts. If the compressors in the station get good attention, there should be no deterioration in the equipment, but it is very difficult to operate. For instance, in operating a group of wells together, it is almost impossible to set the air valves so that each well delivers the proper amount for maximum economy. If air lift wells are operated at different pumping rates, the efficiency falls off. Combination of these two factors may reduce the average efficiency of the plant considerably below that obtained originally.

To summarize, the air lift system is very reliable, has a low first cost, and, if operated at a fairly uniform rate, has a reasonable efficiency.

The turbine pump, on the other hand, has a high efficiency and is very flexible in operation. It lends itself very well to the development of the plant. Opposed to these advantages is the high first cost. The case might arise where a combination of the two would work out very well, an air lift system being operated in conjunction with a steam plant at the steady rate and the peak load pumping rate over the capacity air lift system would be supplied by motor driven turbine well pumps.

THOMAS H. ALLEN:[9] The ordinary old-fashioned bucket pump is so limited in capacity and the up-keep is so high that for water works problems it need not be considered. The popular trend is toward the turbine pump, for it operates with very high efficiency with a consequent low power cost. It has the advantage that it will take the water from any depth below ground and deliver it against a pressure

[9] Memphis, Tenn.

or to a container at any distance at any height. It has the disadvantage that it has to be removed periodically for the renewal of the running elements and the cost of this removal of the pump and the cost of repairs is rather high. No consideration of the air lift can neglect the compressor itself. The air lift for many years was a solution to our deep well problem. It was so simple in its operation, however, that careless rather than careful design, was the custom with the result that many inefficient air lifts have tended to discredit that system of pumping. It has some very distinct advantages. There is absolutely nothing in the well to go wrong. The maintenance is, therefore, low, the first cost is low, and if properly designed, very good efficiencies can be obtained. No consideration of an air lift without consideration of the qualities of the compressor can be considered worth while. In small units with inefficient compressors the efficiency is low; in large units where a number of wells are required with a large volume of water the units can be of such design as to acquire very high efficiency, such as we have in our plant at Memphis. In fact, I should say that for the ordinary small supply where the water is to be taken directly from the well and into the mains, into the tanks or into the reservoirs at a distance, the turbine pump lends itself very well to that kind of a problem. But where the water has to be treated by aeration, which is necessary in handling a great many of the underground waters, the air lift provides a method of pumping which is more reliable, almost as efficient, and far more satisfactory, than is the turbine type of pump. The over all cost in money, taking into account the depreciation of the fixed charges on the equipment and on the wells and the cost of operation, will be less in a large system with an air lift than it will with a turbine arrangement.

MATERIALS AND SIZE OF SERVICE CONNECTIONS[1]

By P. J. Hurtgen[2]

Extensive improvements in the field of water service piping have taken place in recent years. In early years probably the most extensively used service was the galvanized iron pipe with lead gooseneck. Possibly next was the complete lead service. Then we had the lead lined iron pipe, cement lined iron pipe, tin lined iron, wrought iron, and about the last to come within the field is copper service pipe. In selecting service pipe we should consider the following items: Original cost, probable life, past performance, and more or less a guess as to future performance.

The reason for the replacement of galvanized iron pipe by other materials was because of its comparatively short life. The majority of the services in Wisconsin are probably lead, and the more recent installations are either lead or copper. The City of Kenosha in 1925 installed 25 copper services as an experiment. None of these services has caused any trouble. Prior to this time extra heavy lead services were used.

Since 1927, our minimum size service has been ¾-inch, and copper has been used entirely since that time for all ¾-inch services. Services from ¾- to 2-inch inclusive are constructed of lead and from 2-inch up, we use cast iron pipe. Our ordinance provides that all services between the main and the meter must be constructed of either lead, copper, or cast iron pipe.

The department constructs all services between the main and the curb line at the expense of the property owner; and the property owner through his plumber continues the services from this point. It is optional with the plumbers to continue with either lead or copper. The majority of these extensions, however, are made with lead pipe.

A comparison of the cost of copper and lead pipes is shown in table 1.

This price differential, however, does not always prevail, but at the present time it is especially favorable to copper. In 1925, the City of Detroit began experimenting with copper services. Copper pipe

[1] Presented before the Wisconsin Section meeting, August 22, 1930.

[2] Director of Public Works, Kenosha, Wis.

was used for service connections in old dumps, under street car tracks, and other places where lead pipe had caused trouble, and in 1928 the department adopted the use of copper. I make mention of this only to indicate the trend toward the use of a service material that appears to be satisfactory under extraordinary conditions, and can be procured at a reasonably low cost. On the other hand, there are cities that have used lead service pipe for sixty years and are still continuing its use and are entirely satisfied with its performance. My experience with service pipe is confined to galvanized iron, lead, copper, and cast iron pipe. We do not permit the use of galvanized pipe because of its short life. There is no question but that lead and copper are the materials generally used, not only in Wisconsin, but throughout the country on services up to 2-inch and cast iron on the larger services.

TABLE 1

Cost of services

SIZE	COPPER PIPE		LEAD PIPE (EXTRA HEAVY)	
	Weight per foot	Cost per foot	Weight per foot	Cost per foot
inch	*pounds*	*cents*	*pounds*	*cents*
$\frac{5}{8}$	0.418	$10\frac{1}{2}$	$3\frac{1}{2}$	$28\frac{9}{16}$
$\frac{3}{4}$	0.641	$15\frac{1}{2}$	$4\frac{3}{4}$	$39\frac{7}{16}$
1	0.839	$20\frac{9}{16}$	6	$49\frac{1}{2}$
$1\frac{1}{4}$	1.04	25	$7\frac{1}{2}$	$61\frac{9}{16}$
$1\frac{1}{2}$	1.36	33	8	66
2	2.06	50	9	$74\frac{1}{4}$

In our personal experience, with service material, practically all trench work at the present time is constructed with trenching machines and we have noted that in making excavations for sewers or other utility trenches, when we tear into a copper water service pipe, the service usually breaks at the main and most frequently the corporation cock breaks rather than the pipe. This necessitates digging up the service at the main to make the repairs, while, in the case of lead services, the break usually occurs in the trench where no extra digging is required.

SIZE OF SERVICE CONNECTIONS

Service connections should be standardized to meet the requirements of the consumer, making them large enough to give reasonably uniform service. By this I mean, a service should be installed that

will provide the consumer with the amount of water he requires within a reasonable length of time. These sizes, however, should be determined by the water department, based on plans submitted by the consumer, which plans should definitely indicate the number and nature of all outlets to the plumbing fixtures. In Kenosha, prior to the introduction of ¾-inch service pipe, the majority of the service lines accomodating residential areas, were ⅝-inch services; and this size has been found to be entirely satisfactory for the accomodation of such residences as would ordinarily be constructed on a lot up to 60 by 150 feet; also for double 6 room flat buildings provided with the ordinary plumbing fixtures; and with a normal pressure in the distribution system of from 45 to 50 pounds.

When flushometer toilets came into general use, the architects and salesmen specified larger services notwithstanding the fact that in many instances where these larger services were specified the street had been previously paved and the property had been provided with either a ⅝- or ¾-inch service to the curb line, and in order to provide a larger service it would be necessary to tear up the pavement.

When this situation first arose, the water department advised the consumer to make the connection with the ⅝- or ¾-inch service as the case might be, and increase the pipe size from the curb from ⅝- to 1-inch to the meter, and increase this to 1¼-inch. In cases where the curb service was ¾-inch, if it appeared to be necessary, we advised that they continue with a 1¼-inch service to the meter, and continue with a 1½-inch pipe. We then advised that, if the service was later found to be inadequate, we would permit them to cut through the pavement, and install a larger service to the curb line. We have a number of such installations, and in no case have we been called upon to install a larger service, notwithstanding the fact that in several instances we have 2 flushometer toilets, 1 bathtub, 1 sink, and 2 lavatories, connected with a ⅝-inch curb service. It must be remembered, however, that, in making these installations under the circumstances outlined, it is advisable to eliminate as many elbows and short bends as possible in order to provide as direct a flow as possible. We have another installation where, with a 1-inch curb service laid prior to the street paving and also prior to the erection of a large residence, the 1-inch curb connection was continued to the meter, and a ⅝-inch meter was installed, and then continued with a 1½-inch pipe. This service supplies water to 4 flushometer toilets, 3 bathtubs, 1 sink, and 4 lavatories. While there are only three in

this family, they entertain quite elaborately, and no doubt at such times they expect the full capacity of their plumbing fixtures. In the absence of any complaints, we assume that their service is entirely satisfactory. We have another installation where two 1-inch service lines laid to the curb line prior to the paving of the street were later connected and continued into the premises with a 1½-inch service. This line serves 3 flushometer toilets, 2 lavatories, 2 bathrooms, 1 sink, and 1 laundry tub. In addition, it provides automatic sprinkler service for the lawn and gardens, the lot being 120 by 272 feet. The sprinkler system is provided with 33 sprinkler heads and 3 control valves, and with one-third of the sprinkler heads operating at one time it has given entire satisfaction.

As previously stated, I believe the department should determine the size of the service based on plans submitted by the consumer.

Fire line services should be confined to a size smaller than the street main from which the connection is made, and in most instances should not exceed 6 inches.

THE IMPORTANCE OF ADEQUATE FINANCING AND ACCOUNTING IN WATER WORKS PRACTICE[1]

By H. Gordon Calder[2]

Proper financing and accounting are of almost the same importance in any business, but, the larger and more complicated the nature of the business, the more important they become. Many of the principles here set forth apply to all business and because the business of serving the public with water is one which has many functions, including mechanical, economic, commercial, chemical, political and other factors, those principles will apply more particularly to the water business.

If a business is to be successful, three factors must be taken care of, and the success of the business will be in proportion to the efficiency with which those three factors are administered. Those three factors are production, selling, and financing, the last of which includes the accounting function. The three are of equal importance to a business, but without prejudice in favor of my profession, if I had to choose from the three in a business where I had my capital invested to the neglect of the other two, I would unhesitatingly choose financing as the most important.

Many of you gentlemen are engineers. In order to illustrate these factors, I shall use the example of a man who is a consulting engineer, in business on his own account. He is rendering service. That service and knowledge are his production. He must have buyers for that service—obtaining those buyers is selling. And if he spends more for his expenses, rent, advertising, clerical, help, etc., than he receives, it makes no difference how good an engineer he is or how much business he has done, he is losing money. The handling of money is finance.

In examining this factor in business called "finance" let us first look at the financial structure of a business. It is the plan by which

[1] Presented before the St. Louis Convention, June 5, 1930.

[2] Assistant Comptroller, Federal Water Service Corporation, New York, N. Y.

the business is built. Like any other plan it should be followed so far as possible, but should be subject to change where the original plan proves impractical. Any water works, either municipal or private, should make definite financial plans in advance. It should plan the most economical way of getting its original investment and the most satisfactory mode of handling its operating financial requirements. There are engineers who could build a dam, or lay a pipe line, without any plans whatever, because they have had so much *experience* that the plan is in their heads. Men with sufficient financial experience can do practically the same thing. But they usually choose to make their estimates in advance on paper and select whichever method seems most profitable. To do this, they must be familiar with the various methods which might be used. But either engineer or financier must rely on some kind of accounting method to give him the final cost of his completed project.

The accuracy of that accounting is the determining factor of the accuracy of his future calculations. It is possible to keep account on the backs of envelopes and keep them accurately, but it is difficult.

To determine proper policies, the management should have reports showing the status and progress of the affairs of the business. The information may be in the books and files of the office, but the treasurer or auditor of the business should present the important facts about the business in such a manner that they may be readily grasped by others interested in the management. It is worth while before deciding on the type of report to be presented, to investigate the type of report furnished by others and to examine their methods to find whether they are practical for your needs. In making any departure from proven methods, it is wise to get expert opinions, from those qualified by experience to advise. Proper reports are as important to management as charts are to a navigator.

The hope of civilization is that one generation will learn from the mistakes that previous generations have made. The hope of modern business is that we will learn not to repeat past mistakes, and that we will profit by using profitable methods again and again until something better is discovered. Business history is kept in the ledger, and that business history in many cases is just like Civil War history. One story is taught in the North and another is taught in the South. There are some broad minded and well informed educators who teach both sides impartially and lucky is the student who learns his civil war history from such a professor, and lucky is the business which has

an accounting system which presents the true history of that business, disregarding predetermined beliefs. Guessing can largely be eliminated from such a business.

Businesses are run according to the policies of the management, either expressed or otherwise, and determining policies for the management of a water plant without accurate accounting information is like buying a grab bag at a church fair—you do not know how good it is until you have closed the deal and its too late to back out. In the water business you know when you get your bank statement—the notice that your notes are due—or that your warrants are due and you cannot sell your bonds. The only difference is that, with the grab bag, the suspense is soon over. There is a water business with close to a million dollars revenue a year which is nearly thirty years old and has never paid a dividend, due entirely to improper financing and fallacious policies based on erroneous accounting. The engineering has been satisfactory and the growth of the plant has been ideal for a profitable venture, but the forgotten factor beat them.

So much for theory. One of the strongest arguments for proper accounting is that in many instances modern methods of handling records actually save money in the operation of the office itself.

EXAMPLES OF IMPORTANCE OF FINANCIAL POLICY

Here are a few concrete cases of the principles we have been discussing. A certain water plant was acquired by Company A. The construction accounts were in miserable shape and had not been transferred to the fixed capital accounts for five years. Upon analysis, an amount in excess of $20,000, representing work which was for the account of an outside company, was buried in these construction accounts. It was billed (after being lost for two years) to the consumer and collected. The total cost of that analysis was less than $500. In another case an audit of a company disclosed an error in accounting which made something over $17,000 available for dividends which would have otherwise been closed out in its capitalization. You have all read of refunds on federal taxes. You may be sure that in most cases they called for expert accounting analysis before they were secured.

There was a merchant in the lumber and grain business whose credit limit at the local bank was $3,000. The records which he kept of his business were negligible. He could never furnish his bank with a complete statement of his financial condition or of his profit and loss.

As a result of the limit of his credit at the bank, he frequently lost discounts on invoices which ran as high as 25 percent. He finally called in a bookkeeper. The bookkeeper put his records in shape and furnished him with monthly statements, copies of which were mailed to the banker. The banker as a result of receiving definite information, and as a result of the knowledge that this man was keeping records of his business, increased his credit limit to $10,000 and then to $20,000 with the result that the merchant made the difference between 25 and 5 percent on $17,000 annually. After three years of keeping books he discontinued the grain business because his records showed that it had become unprofitable. His business and his personal financial standing improved amazingly through definite knowledge of his affairs. When an issue of bonds for a water works is submitted to a banker, regular statements and proper accounting may mean the difference between 5 and 7 percent in interest paid on the bonds and may also be the difference between being able to issue $500,000 or $1,000,000 of bonds.

Your division of Finance and Accounting was organized last June in Toronto to present the latest methods and ideas of financing and accounting to the Association. May I urge you all again to help this new division of the American Water Works Association.

DISCUSSION

L. M. ANDERSON:[3] It is the exception when a comprehensive financial program is outlined at the time of the original construction of a water works system.

Water works as a rule have a lowly birth.

Very often in a village neighbors will prevail upon the one fortunate enough to own a good well, equipped perhaps with a windmill, pump, and tank to furnish them with a water supply.

The water business will grow with the community and it will not be long before the one supplying the water will find that he is in a utility business under the jurisdiction of some regulating body. This situation may be very embarrassing and about the only solution will be to form a Mutual Water Company of the consumers or sell to some individual or company that wants to be in the water business.

When a municipal corporation, whether large or small, proposes to install or purchase a water works system, it should consider not only

[3] Controller, Department of Water and Power, Los Angeles, Calif.

the original cost in the financial plan, but also provide in that plan for future extension and betterments.

Whenever the laws under which it is operating will permit, it is advisable that the body responsible for the construction and operation of the water works have the authority to adopt its own financial program, whether the funds are to be provided out of revenue or by bond issues. If such matters are left to some other official body or it requires a vote of the people to secure funds by bond issue for utility purposes, there is no telling what may happen.

It has been my observation that it is easier to secure money for some sentimental purpose than for a practical or necessary one, if left to the popular vote to decide.

It is next to impossible at the inception of a plan for water works to forecast what the requirements will be eventually. Here in Southern California no one can tell what the demands will be for water in a few years hence.

Communities and business centers will grow from a few hundred to many thousands in a very short time and both the supply and distribution are serious problems.

The money to meet the required expenditures to care for the growing demand for water should be provided by long term bonds.

Efficient service is demanded and should be provided for in an assured water supply, ample capacity in the way of storage, pumping machinery and distributing system, all of which requires no small amount of money. It is not right or equitable to burden the present rate payer with the heavy load caused by the necessity of keeping pace with rapid expansion.

Unfortunately when the laws of some of the states were adopted the legislators had in mind governmental requirements only. The matter of financing a public utility was not considered. Difficulties, the result of lack of proper utility laws, are being corrected and in time will be ironed out and the responsible body charged with water works management can carry on without interference.

Now, how can construction of water works be financed?

May I suggest that the first step necessary is to secure proper legislation (if the right laws are not already on the statute books) that will place the management in an independent non-political body divorced entirely from the city government; also legislation that will legalize the issuance of long term bonds, not serial ones payable in installments, but bonds that can be paid off at some future date by

the issuance of new bonds or cash. The bonds should be authorized and issued by the Water Commission.

If there is a demand for a water works the rates should be ample to pay operating expenses, depreciation charges and bond interest.

All construction properly chargeable to capital accounts should be paid for out of bond funds. If there is a surplus from operations, it should be placed in a sinking fund for the redemption of bonds when due.

Nothing new has been advanced in the above suggestions, but attention has been called to the simple method by which wisely managed corporations finance their utility business.

ACCOUNTING METHODS

There should be no argument as to the necessity of comprehensive and reliable accounting records, without which no business can succeed.

The California State Railroad Commission has prescribed what is known as a "Uniform Classification of Accounts for Water Corporations." This classification is complete in detail and any accountant can readily follow it as almost every possible transaction is defined. Under this classification water corporations are divided into three classes, as follows:

Class A. Corporations having an average annual operating revenue exceeding $100,000.00.

Class B. Corporations having an average annual operating revenue of less than $100,000.00, and more than $25,000.00.

Class C. Corporations having an average annual operating revenue less than $25,000.00.

Classes B and C, are not required to keep the more extended divisions of any or all of the accounts prescribed, but may do so if they wish.

It is suggested that they keep these accounts, as all persons or corporations, other than municipal, in the water business must file a yearly report with the Railroad Commission.

The Department of Water and Power of the City of Los Angeles has adopted the Railroad Commission's classification of accounts, although not required to do so, but for good business practice.

F. W. SCHULZ:[4] The subject of proper financing and accounting is

[4] Comptroller, Community Water Service Company, New York, N. Y.

so vast because of its many phases, that an attempt to cover it thoroughly within the limits of a short discussion, is impossible.

Financing today is no longer a hit and miss proposition, but is becoming something of an exact science. When a corporate structure is in need of funds or capital, the procedure as a rule, is closely supervised by bankers, trustees and similar officials. Invariably audits are demanded by fully qualified accountants and the best legal talent is employed. The restrictions are many and varied, and great stress is laid on values, on actual and potential earning power and on such considerations as efficient management, prospects for future growth, etc.

Proper and adequate financing is largely contingent upon the condition of the accounting records, and it is logical to assume that, if the records are in poor condition, poor financing will result. For this reason, this discussion will be confined to the necessity for true and complete accounting records, rather than to the actual mechanics of financing.

In the past ten years public utility executives have come to attach increasing importance to accounting. Absolute accuracy is demanded in the accountant's work, as it has a direct bearing on the relations of the company and its consumers. Perhaps nothing is more advantageous to a company, so far as public opinion is concerned, than accurate accounting, as it results in the presentation to all interested of a composite picture of the company's activities and shows as well the reasonable basis on which the charge for services rendered is founded.

Whenever rate increases are applied for it is generally necessary that the utility company's accounting records and practice undergo a searching investigation. In such cases the books and records may be judged largely by a few examples brought out in cross-examinations. Errors are often exaggerated in importance, especially if they happen to be, or can be construed as errors detrimental to the consumer.

Mr. Calder stated in his paper that "proper reports are as important to management as charts are to a navigator." This statement cannot be over-emphasized, and, in order to compile and present these reports properly, it is obvious that the accuracy of the books of accounts and other records is of great importance. Furthermore, to be of maximum benefit the presentation must be logical in arrangement and on a basis which will show by comparisons, the trend of

both revenues and expenses. Without such information certain methods of operation may be causing the company to suffer losses of which the management will not be aware until the amounts have become very serious.

There follow a few illustrations which point out some of the evils that have resulted from the lack of sufficient records, and the absence of accurate accounting methods.

A water company was organized in 1908 and established rates at the date of incorporation. The records that were kept were untrue and worthless, due to accounting ignorance. The directors were under the impression that the company was building up a surplus and were continuing to declare dividends.

Some time during 1914 a Board of Public Utility Commissioners was organized to control the operation of utilities in the state in which this company operated. This forced the company to keep its accounts and records as prescribed by the Commision and necessitated the employment of a competent accountant. After one year of operations with proper accountant, the books showed a deficit of approximately $8,000 for the year. This deficit was not due to the additional expense occasioned by the accountant's salary, as the corrected books disclosed a deficit of $60,000 applicable to previous periods.

It was then realized that an increase in rates was necessary, and after much wrangling and expense, an increase was granted, which added approximately $20,000 annually to the company's surplus. Had true accounting records been maintained in years prior to the creation of the Commission, rates could doubtless have been adjusted as required and the company would have been spared a deficit and an impairment of its capital.

A few years back a purchase investigation was made of a water company owned by an individual whose original investment amounted to $200,000. His common boast was that he kept no books other than a cash record, and that he had built up a wonderful business without them. He was offered $300,000 for the property and promptly accepted, thinking he was making a profitable trade. The investigation disclosed that the actual property valuation at cost, disregarding good will and intangibles, was in excess of $400,000. His idle boast, based on ignorance of the value of accounts and records, had cost him $100,000.

One of the most common errors which lack of appreciation of the

importance of proper accounting led to, is illustrated by a case which recently came to the writer's attention. A company owned for some years by the members of a family, never seemed to be able to pay any dividends. An examination by a qualified accountant in connection with a purchase of the property, disclosed the fact that the owners did not understand the distinction between capital expenditures and operating charges. As a result, they had for many years, charged extensions to operations which depleted their surplus so that no dividends could be paid, and at the same time their books did not show in the Capital account the increased value which was constantly going into the plant. What had always been to them more or less of a "white elephant," was, in reality, a plant capable of earning a good return for its stockholders.

These examples illustrate clearly the principle, that it is good business to follow this plan, "Establish the best and most practical accounting system required for your needs, and employ a competent accountant to keep your records accurately."

REPORT OF COMMITTEE ON PURIFICATION DATA AND FORMS[1]

The Committee on Purification Data and Forms recommends that filter plant operators make their regular monthly reports of plant operation on forms similar to that submitted herewith, and that the following suggestions be given careful consideration:

1. The entire monthly report should be made on a single sheet exactly 11 inches wide and of sufficient length to meet all requirements. The reports can then be folded to 8½ by 11 inches size for binding or filing.
2. A 2-inch binding margin should be left on the left hand edge, and at least ¼-inch margin be left on the top, bottom, and right hand edges.
3. The report should be printed or reproduced on thin paper or tracing cloth, so that blue prints can be made.
4. No printing should be put on the back of the sheet.
5. Title should be placed in the upper right hand corner.
6. Remarks should be made in the space to the left of the title.
7. "Milliliter" should be substituted for "cubic centimeter."
8. In filling in the forms under the head of B. Coli, the fractional form be used, the denominator indicating the number of tubes tested, and the numerator indicating the number of tubes positive.
9. In making annual reports the same general arrangements should be used, but the report should be separated into two sheets, grouping the physical and chemical data on one sheet, and that relating to microörganisms and bacteria on another sheet. The spacing and lettering should be larger than on the regular monthly report so that photographic reproductions can be made. Monthly averages should be reported. Maximum and minimum results should be for the maximum and minimum day only.
10. If Soda Ash, Ammonia, Copper Sulphate, etc. are used, they should be reported in the position indicated, and where

[1] Committee of the Water Purification Division.

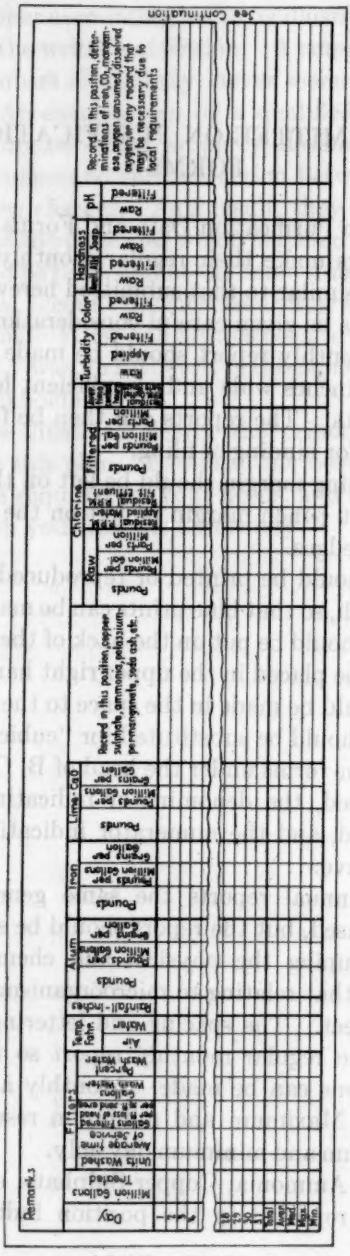

See Continuation
Remarks
Day
Million Gallons Treated
Units Washed
Average Time of Service
Filters
Gallons Filtered per ft. sq. of sand area
Gallons Wash Water
Percent Wash Water
Temp. Water
Air
Rainfall - Inches
Alum
Pounds
Pounds per Million Gallons
Grains per Gallon
Iron
Pounds
Pounds per Million Gallons
Grains per Gallon
Lime - Soda
Pounds
Pounds per Million Gallons
Grains per Gallon
Record in this position, copper sulphate, ammonia, potassium permanganate, soda ash, etc.
Raw
Pounds
Pounds per Million Gallons
Parts per Million
Chlorine
Residual P.P.M. Applied Water
Residual P.P.M. Filter Effluent
Filters
Pounds
Pounds per Million Gallons
Parts per Million
Turbidity
Raw
Applied
Filtered
Color
Raw
Filtered
Hardness
Raw
Filtered
pH
Raw
Filtered
Record in this position, determinations of iron, CO2, manganese, oxygen consumed, dissolved oxygen, or any record that may be necessary due to local requirements

FIG. 1

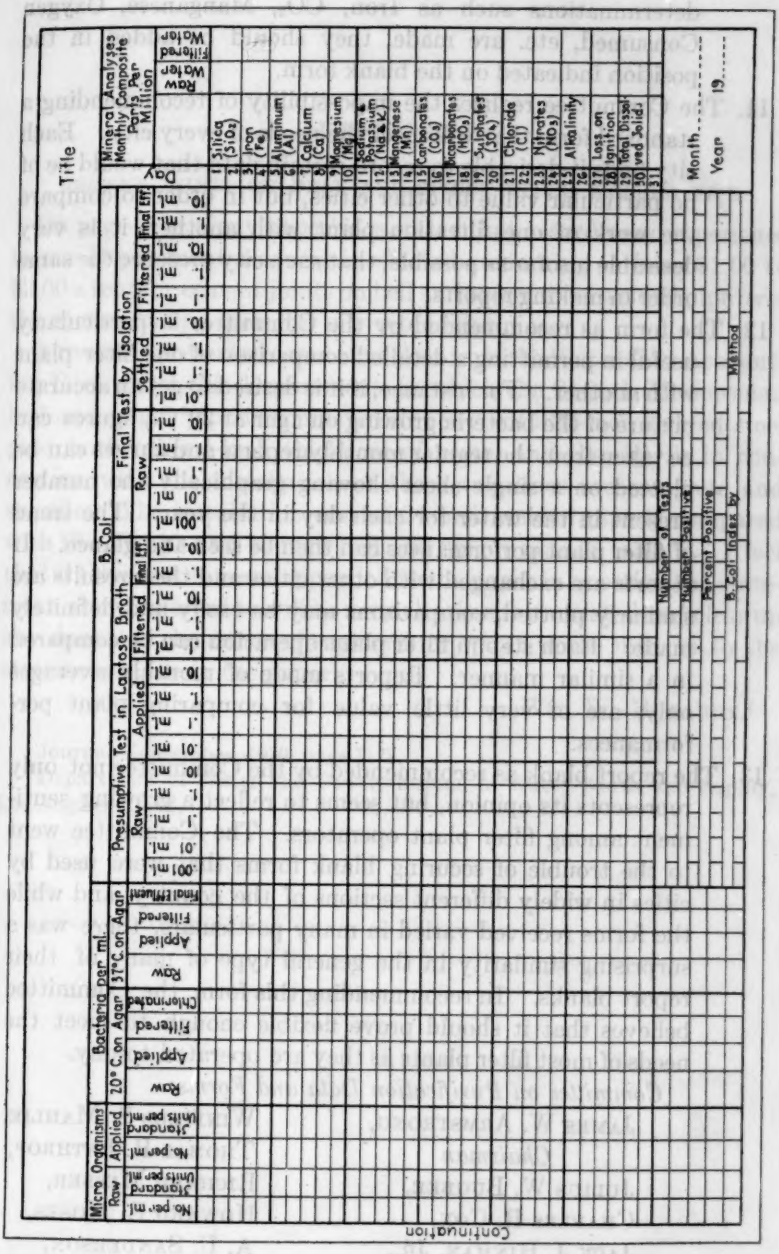

Title
Minera Monthly Composite
Analyses Parts Per Million
Raw Water
Filtered Water
Day
1 Silica (SiO2)
2
3 Iron (Fe)
4
5 Aluminum (Al)
6 Calcium (Ca)
7
8 Magnesium (Mg)
9
10
11 Sodium & Potassium (Na & K)
12
13 Manganese (Mn)
14
15 Carbonates (CO3)
16
17 Bicarbonates (HCO3)
18
19 Sulphates (SO4)
20
21 Chloride (Cl)
22
23 Nitrates (NO3)
24
25 Alkalinity
26
27 Loss on Ignition
28 Total Dissolved Solids
29
30
31
Micro Organisms Raw
Applied
No. per ml.
Standard Units per ml.
No. per ml.
Standard Units per ml.
Bacteria per ml.
20° C. on Agar
Raw
Applied
Filtered
Chlorinated
37°C. on Agar
Raw
Applied
Filtered
Final Effluent
B. Coli
Presumptive Test in Lactose Broth
Raw
.001 ml.
.01 ml.
.1 ml.
1. ml.
10. ml.
Applied
.001 ml.
.01 ml.
.1 ml.
1. ml.
10. ml.
Filtered
.1 ml.
1. ml.
10. ml.
Final Eff.
1. ml.
10. ml.
Final Test by Isolation
Raw
.001 ml.
.01 ml.
.1 ml.
1. ml.
10. ml.
Jelfied
.01 ml.
.1 ml.
1. ml.
10. ml.
Filtered
.1 ml.
1. ml.
10. ml.
Final Eff.
1. ml.
10. ml.
Number of tests
Number Positive
Percent Positive
B. Coli Index by
Method
Month.....
Year 19.....
Continuation

FIG. 2

determinations such as Iron, CO_2, Manganese, Oxygen Consumed, etc. are made, they should be added in the position indicated on the blank form.

11. The Committee realizes the impossibility of recommending a standard form that will meet the needs of every city. Each city finds it desirable to record certain data that would be of no particular value to other cities, but in order to compare the work of one filtration plant with another, it is very desirable insofar as possible, that each city preserve the same order in making reports.

12. The form as recommended by the Committee is particularly useful in permitting a detailed comparison of one filter plant with another. For instance, if it is desired to get an accurate picture of the bacteria growing on agar at 20°C., figures can be taken from the regular monthly reports and curves can be plotted on a single sheet showing graphically the number present in the water for each day in the year. The trend of filter plant performances can then be seen at a glance. If reports are exchanged with other cities and their results are similarly plotted, comparisons may be easily and definitely made. Each step in filter plant operation can be compared in a similar manner. Reports made of monthly averages only, are of very little value for comparing plant performances.

13. The report blank as recommended by the Committee not only represents its opinion, but seems to reflect a growing sentiment among filter plant operators. The Committee went to the trouble of securing blank forms that were used by cities in widely different sections of the country, and while the forms received varied in many particulars, there was a surprising similarity in the general type of many of their report blanks. In recommending this form, the Committee believes that it should prove flexible enough to meet the needs of most filter plants as they are operated to-day.

Committee on Purification Data and Forms:

JAMES W. ARMSTRONG, *Chairman*
JULIUS W. BUGBEE,
CHARLES R. COX,
JACK J. HINMAN, JR.,
HARRY E. JORDAN,
WINFIELD S. MAHLIE,
THOMAS R. LATHROP,
RICHARD MESSER,
HOWARD E. MOSES,
A. U. SANDERSON,
WM. M. WALLACE.

DISCUSSION

CEMENT JOINTS ON BELL AND SPIGOT STEEL WELDED PIPE

On page 1179 of my paper[1] it is stated that according to experience in our Department a saving is effected amounting to from $1.00 to $2.00 a foot for cement joints on bell and spigot steel welded pipe over the corresponding diameter of riveted joint pipe.

A record is presented in table 1 taken from the Water Cost Accounting Division of this Department, in which the 24-inch Mannesman bell and spigot pipe is compared directly with 24-inch riveted steel pipe, both being $\frac{1}{4}$-inch thickness of plate, and the details in connection with the installation of these two lines are complete and specific. The installation cost of 24-inch welded pipe is compared with 18-, 24-, 30- and 40-inch bell and spigot pipe in table 2. I feel that if these exhibits are appended to the paper in question the statement above referred to will be very clearly substantiated and clear up any doubts in the minds of engineers or water works men as to the statement made in the paper.

WM. W. HURLBUT.[2]

[1] Journal, September, 1930, page 1178.

[2] Department of Water and Power, Bureau of Water Works and Supply, Los Angeles, Calif.

TABLE 1

Comparative Report: Costs,—24-inch Mannesman Pipe vs. 24-inch Steel Riveted Pipe

24-INCH MANNESMAN PIPE (AUTH. 1900-S)		24-INCH STEEL RIVETED PIPE (AUTH. 2190-S)	
Location: Pvt. R/W El Sereno Ave. from 499 feet S C/L Ascot Tower to Alhambra Ave.		Location: Avalon Blvd., Main to 97th Sts., and Wadsworth.	
Supt. Pipe Construct.: J. T. Lyman L. A. Ghiotti		Supt. Pipe Construct.: J. T. Lyman L. A. Ghiotti	
Foreman: Palmersheim, Terrel.		Foreman: Palmersheim, F. Brooks.	
Installed 4699 feet—24-inch Bell & Spigot Steel (Mannesman Pipe).		Installed: 6770 feet—24-inch x ¼-inch Steel Riveted Pipe.	
Digger and Backfiller Used. Dirt Sts. and R/W.		Digger, Backf., Hoist, Air Comp. used. Oil, Oil and Gravel, Paved Sts.	
Started: 7-23-26.		Started: 8-23-26.	
Completed: 8-9-26.		Completed: 10-18-26.	
Cost Detail:		*Cost Detail:*	
Labor	$1,090.00	Labor	$5,958.50
Equipment	626.00	Equipment	1,954.51
Material	19,685.52	Material	28,958.62
Transportation	183.70	Transportation	1,214.52
*Resurfacing		Resurfacing	
Direct charges	$21,585.82	Direct charges	$38,086.15
†Indirect charges	1,015.90	Indirect charges	3,192.58
Total Cost	$22,601.72	Total Cost	$41,278.73
Cost Analysis:		*Cost Analysis:*	
Total cost per foot installing 24-inch Mannesman	$4.81	Total cost per foot installing 24-inch Steel Riveted	$6.10
Per foot, Labor and Equipment only	$0.36	Per foot, Labor and Equipment only	$1.17
Field Conditions: Fair Average condition throughout—no traffic or substructure interference.		Field Conditions: Some traffic and sub-structure interference, necessary to backfill and clean up job during progress of work, take care of driveways, etc.	

* Resurfacing: To permit of comparison no resurfacing changes figured on either job.

† Indirect charges: Figured on both jobs, following basis:

Tool Expense	4 percent Labor
Engineering Expense	5 percent Labor
Plant Supervision	15 percent Labor
Sat. time allowance, vacation, etc	15 percent Labor
Warehouse Expense	3 percent Material

TABLE 1—*Concluded*

Equalizing costs for comparative purposes a credit of 15 cents per foot should be allowed Steel Riveted job for added labor and equipment charges owing to the above differences in field conditions, with a final result as follows:

Cost reduction to this Department, installing Mannesman vs. Steel Riveted:

$1.14 per foot, or a total added cost on the Steel Riveted Job of $7,717.80

Remarks:

Mannesman Pipe purchased from M. Allen Co., $3.98 per foot FOB Trenchside.

Steel Riveted Pipe purchased from Llewellyn Iron Works, $3.80 per foot FOB Trenchside.

Neither of the above jobs were done under pressure and therefore permit of comparison.

Thickness: Steel Riveted Pipe—$\frac{1}{4}$-inch (trifle light); Mannesman—$\frac{1}{4}$-inch (heavy).

TABLE 2

Cost comparison—Bell and Spigot Welded Steel Pipe

LOCATION	SIZE	FOOTAGE	(LABOR AND EQUIP.) COST PER FOOT	DATE COMPLETED	REFERENCE NUMBER
	inches		*dollars*		
Moorpark Tujunga Ave.	24 (Welded)	10,585	1.26	1–31–30	9600-S (VN)
Chautauqua Blvd. Bestor Blvd. R/W N. of Beverly	18 (B/S)	3,459	0.70	12–26–28	7783-S (LA)
Humboldt St. Avenue 19 San Fernando Rd.	24 (Mann. B & S)	4,976	1.25	4– 1–28	2047-GW (LA)
Alhambra to Tower So. of Ascot Res.	24 (Mann. B & S)	4,699	0.36	8– 9–26	1900-S (LA)
Riverside Dr. Knox to Dayton	40 (B/S Steel)	1,158	0.88	5– 7–29	8748-S (LA)
Riverside Dr. Knox to Dayton	40 (B/S Steel)	1,786	0.85	4– 3–29	8747-S (LA)
Riverside Dr. Knox to Dayton	40 (B/S Steel)	3,852	0.81	5–25–29	8746-S (LA)
Riverside Dr. Knox to Dayton	40 (B/S Steel)	2,446	0.89	4–10–29	8358-S (LA)
(Total 40 inch only—9,242 feet, average cost per foot $0.85)					
Montana Ave. Sepulveda Blvd.	30 (B/S Steel)	2,966	0.94	2– 3–28	5832-S (LA)

ABSTRACTS OF WATER WORKS LITERATURE[1]

FRANK HANNAN

Key: American Journal of Public Health, 12: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the Journal.

Water Works Intakes at Hamilton, Ont. H. S. PHILIPS and JAMES STODART. Contract Record and Eng. Rev., 43: 241–4, March 6, 1929. Water supply of Hamilton orginated in 1859, the source then being infiltration basins on the shore of Lake Ontario. Capacity of basins gradually diminished and in 1889 an intake was laid connecting the basins with the lake. A second 48-inch intake was constructed in 1913; and in 1923, a short, but almost total interruption of the city supply occurred as a result of this latter intake becoming blocked with ice. To avoid further trouble, a new 60-inch steel intake was constructed in 1926, the infiltration basins being abandoned and a conduit installed connecting the new intake and the pumping station. Construction of new intake is described in detail. The pipe extends above 3000 feet into the lake and terminates in an intake crib consisting of an interlocking steel sheet piling shell, 22 feet in diameter, in centre of which a steel cone is set in concrete, connected to the pipe by a cast iron elbow. Reinforced concrete baffle plate anchored above the opening prevents surface water from being drawn into the intake. The 6 intake ports have area of 54 square feet, giving velocity of 1.45 feet per second at time of maximum draught. Ultimate velocity in pipe will be 4.7 feet per second. Pipe is set in concrete for distance of 430 feet from shore. Longitudinal seams are double riveted and circumferential, single riveted. Five cast iron flexible joints were provided. Before entering pumping station the water passes through travelling screens, housed in the same building as the chlorination equipment. The total consumption in 1927 was 6,420,691,500 gallons, equivalent to 138 gallons per capita per day.—*R. E. Thompson.*

Atchafalaya Floodway for Mississippi Flood Control. W. W. DEBERARD. Eng. News-Rec., 103: 492–5, September 26, 1929. Details given of the Atchafalaya floodway and its function.—*R. E. Thompson.*

Water Works Pumphouse in Montreal Will Have Unusual Architectural Design. Contract Record and Eng. Rev., 43: 123–4, February 6, 1929. Brief description of new McTavish pumping station. Owing to high-class surround-

[1] Vacancies on the abstracting staff occur from time to time. Members desirous of coöperating in this work are earnestly requested to communicate with the chief abstractor, Frank Hannan, 285 Willow Avenue, Toronto 8, Ontario, Canada.

ings and residential character of district in which it is located, a pleasing Chateau design was adopted. Details are given of distribution system districts and of reorganization of same to incorporate system of the Montreal Water and Power Co., recently acquired. New station will be used for repumping water to high-level zones. A brief outline of the pumping equipment is appended. The 6 pumping units, 3 of 12 m.g.d. and 3 of 15 m.g.d. capacity, will be of the two-single-stage-pumps-in-series type and will be driven by self-starting synchronous motors. Suction lines (from adjoining reservoir) will be controlled by hydraulically-operated butterfly valves and discharge line of each pump will be controlled by a JOHNSON combination stop and check valve. These valves will be operated automatically in case of power interruption by means of special relay so as to close three-quarters way in about 2 to 3 seconds and remainder in predetermined time, e.g., 5 to 10 seconds, thus eliminating water hammer. Fields of synchronous motors will be automatically opened in case of power interruption so that, if any pump should turn in the reverse direction before the JOHNSON valve is closed, the motor will not act as generator.—*R. E. Thompson.*

Keeping Concrete Pipe Damp During Curing. W. F. SHAW. Eng. News-Rec., 103: 430, September 12, 1929. Method found best consists of standing pipe sections on end close together on curing ground (fine gravel and cement with some drainage) and covering with burlap, edges well overlapping, reaching to ground on all sides, and weighted down. In this way pipe is completely enclosed and no air can circulate. Burlap is sprinkled by hand.—*R. E. Thompson.*

Water Pipe Practice at Detroit, Mich. GEORGE H. FENKELL. Eng. News-Rec., 103: 410, (1929). During the last few years Detroit has employed steel pipe almost exclusively for mains larger than 36-inch, principal reasons being the favorable cost and the fact that steel pipe is not subject to sudden failures. In recent contract proposal bids have also been invited on pipe of pure Fe, or of pure Fe plus a small percentage of Cu. In evaluating the bids, 10 years longer life was allowed for latter pipe than for steel, the values assumed being 45 and 35 years respectively. The water supply, derived from the Detroit River, has a hardness of about 100 p.p.m. and is non-corrosive. The soil, in general, is also of a non-corrosive nature.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

Cast Iron Pipe of New Type Now Made in Canada. Contract Record and Eng. Rev., 43: 119–22, February 6, 1929. Illustrated description of new works of Gartshore-Thomson Pipe and Foundry Co., Hamilton, Ontario, for manufacture of cast iron pipe by "sand spun" pressure process. Advantages of sand spun pipe are detailed. Tensile strength is 38 percent higher than that of pit cast pipe and wall section is uniform throughout, inside and outside diameters being perfectly concentric. As sand mould is retained in process, pipe possesses true gray iron structure, with its desirable characteristics. It is as flexible as the older type and, in addition to greater bursting strength, is also better able to resist such stresses as shock and bending.—*R. E. Thompson.*

Water Meters are Complete Cure for Wastage. W. A. McLEAN. Contract Record and Eng. Rev., 43: 122, February 6, 1929. Brief general discussion of water consumption and waste. Metering is remedy for waste. In absence of metering, house to house inspections should be made.—*R. E. Thompson.*

Prevention of Percolation through Dams. JOSEPH A. KITTS. Eng. News-Rec., 103: 430, 1929. Discussion of paper by N. A. BOWERS. The factors affecting the quality of concrete are discussed, with particular reference to technical control and workmanship. The need of a pozzuolanic constituent in concrete to convert the free lime to Ca silicate is pointed out.—*R. E. Thompson (Courtesy Chem. Abst.).*

Town Penalized for Failure to Chlorinate. Contract Record and Eng. Rev., 43: 1114–6, 1118, September 18, 1929. In spring of 1927, Kingsville, Ont., a town of about 2400 population, suffered an epidemic of typhoid fever, there being about 50 cases of which 10 or more proved fatal. Recent court decision awarded $2000 damages in suit brought by a relative of one of the deceased against the town. Text of decision is given and the conditions leading up to the epidemic are described. Question of sewage disposal and water supply had been problem for many years. There is no sanitary sewer system and connections had been permitted between septic tanks and cesspools and the storm sewers, which ultimately discharge into Lake Erie, the source of water supply. Since 1917 the Ontario Department of Health have endeavoured to compel the municipal authorities to improve the sanitary conditions of the town and chlorinate the water supply. Pressure filters were installed in 1921 and in 1926 it was decided to construct a new intake. Old intake, however, failed in February, 1927, before new intake was completed. An opening was made in settling basin and trench dug connecting basin with the lake. Water thus obtained was passed through the plant into the distribution system, a notice being published in a local paper advising boiling of all drinking water. Ice disappeared from lake prior to March 24, and first case of typhoid developed on April 1. Systematic chlorination was adopted on April 14 and the epidemic abated. New intake has since been completed and storm outfall sewer discharge is now being chlorinated.—*R. E. Thompson.*

Tests on Concrete Made at Coolidge Dam. BRUCE JOHNSTON. Eng. News-Rec., 103: 456–7, 1929. A very slight variation in certain factors may cause a variation in strength equal to that brought about by a relatively large change in water-cement ratio. Tests have indicated the great importance of the following factors: chemical composition of the mixing water; presence of fine material in the sand; and order in which ingredients are mixed. The spring water used during construction of dam, which had total solid content of 1460 p.p.m., gave 48.9 percent greater strength than Phoenix city water. Removal of the silt from the sand and coarse aggregate reduced the strength 41.9 percent as compared with unwashed aggregate. Addition of part of the mixing water to the gravel prior to adding the cement resulted in an average increase in 7-day strength of 13.8 percent and a maximum increase of 40.3 percent. The latter is believed to be due to the dry cement adhering to the wet rock and

forming a rich mortar which enters the rock pores and increases the bond strength. Further tests showed an increase in strength of 18 percent at 28 days resulting solely from the use of pre-wet aggregate. During construction of the dam the aggregates were wet down daily.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

Translating Theory into Practice in Regional Engineering. JAY DOWNER and JAMES OWEN. Eng. News-Rec., 103: 443–8, September 19, 1929. The water supply of Westchester County is interlocked with that of New York City, as most important sources have been developed by city. Region is now served by 15 municipally-owned plants and 11 privately-owned corporations. Sanitary protection of watersheds is becoming increasingly difficult and land values will soon force utilization of vacant watershed areas for other purposes. Present trend is to purchase water from city. Agreement has been reached on price of $100 per m.g. for Catskill water and $66.66 for Croton water, latter being under lower head.—*R. E. Thompson.*

Standing-Wave or Hydraulic-Jump Flumes Again. CLEMENS HERSCHEL. Eng. News-Rec., 103: 431, 1929. Discussion of paper of C. C. INGLIS, comparing the relative merits of standing wave flumes and venturi meters.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

Montreal Water Purification Works. Contract Record and Eng. Rev., 43: 347–8, April 3, 1929. The original plant, completed in 1918, consisted of 150-m.g.d. low lift pumping station, 16 pre-filters and 16 final filters with nominal combined capacity of 50 m.g.d., and 6¼ m.g. clear well. Cost of plant, exclusive of land, was $1,408,300 or $28,000 per m.g.d. The satisfactory results from the existing pre-filters and the comparatively low cost of constructing 32 additional filters resulted in adoption of this plan in recent extensions. The 32 additional filters have a nominal capacity of 100 m.g.d. The original pre- and final filters will be operated as a combined plant and the new filters will be operated as rapid sand units. The cost of the extensions was $1,009,140 or $10,000 per m.g.d. New filters were designed for air and water wash. The raw water is drawn through one of two intakes from the St. Lawrence River about 20 miles below the mouth of the Ottawa River, and conveyed to the plant by the Montreal aqueduct canal. Latter is 5 miles in length, the lower 3 miles functioning as a settling basin. The quality of the raw water depends on the relative proportions of the two river waters, the Ottawa water being more colored and requiring 7 times as much chlorine for sterilization as that of the St. Lawrence. Data on the bacterial removals effected by the original plant are included. The efficiency of the rapid filters, when operated at an average rate of 139 m.g.d. without coagulant, was 75 percent, and of the final filters, when treating a mixture of 3 parts of raw and 4 parts of pre-filtered water at an average rate of 11.5 m.g.d., was 85 percent, the overall efficiency (before chlorination) being 87.5 percent.—*R. E. Thompson.*

Saving Money on Trench Work. Contract Record and Eng. Rev., 43: 972–4, August 21, 1929. Comparative cost data given for hand and pneumatic

pavement breaking operations, latter showing great savings. Where no rock is encountered it has been found possible to drive service pipe directly through earth by means of pneumatic tools, saving the breaking of sidewalks, etc. Average saving has been found to exceed $12.00 per service. In some cases pipe has been driven 60 feet, average reported by one company being 28 feet and time of driving about 35 minutes. Considerable savings can also be effected by use of a pneumatic machine for tapping mains.—*R. E. Thompson.*

Chemical Treatment of Hydraulic Dam Cores. F. E. HANCE. Eng. News-Rec., 103: 542, 1929. Research on soil materials available for dam construction in Honolulu disclosed the fact that many of the mountain sandy loams or silts were highly acid in reaction and, when agitated with water, formed a semi-permanent dispersion without imparting their acidity to the water. On neutralization with soda ash the soil immediately flocculates, forming a well-defined granular deposit of greater permeability than the original soil. If, however, sufficient alkali is added to give a pH value of approximately 10, the precipitated material is of the nature of fat clay and is 200 to 1000 percent more impermeable, the voids being reduced to a remarkable extent. The application of the principle to the construction of dams is suggested and is being tried out.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

Construction Ingenuity Displayed at Diablo Dam. Eng. News-Rec., 103: 320–4, August 29, 1929. Diablo dam on Skagit River in Washington is being erected as part of comprehensive water power development being carried out by the city of Seattle. Structure is constant-angle arch between gravity-section abutments and has remarkable height of 425 feet. Reservoir will have capacity of 90,000 acre-feet and drainage basin of 1100 square miles. Construction methods are described and illustrated in some detail. Belt conveyors swung from hoist towers in manner of gravity chutes are placing a dry, lean mix, strength obtained being remarkably high. Another feature is the use made of grouting to form subsurface cofferdam walls in the permeable boulder-gravel bottom of the river so that dam foundation could be laid dry for excavation.—*R. E. Thompson.*

Is the Water-Cement Ratio the Last Word? NATHAN C. JOHNSON. Eng. News-Rec., 103: 471, 1929. It is pointed out that there are many problems for which the water-cement ratio offers no solution or clue. Separation and lack of uniformity in mix because of the wash-out action of too much mixing water is as important as reduction in strength.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

Town Water Supply Fully Metered. S. L. VAN NORDEN. Eng. News-Rec., 103: 347, August 29, 1929. Water supply of Glen Ridge, N. J., is obtained from 60-inch feeder line connecting Cedar Grove distributing reservoir with City of Newark. Distribution system is fed by gravity from 3 points on feeder, amount withdrawn being recorded by meters. Town is completely metered, consumers receiving water at cost. Meter practice is outlined. Less than one percent of meters tested have shown error greater than 2 percent. The error in more than half of these was in favor of the consumer.—*R. E. Thompson.*

Lloyd Dam in India, the Largest in the World. REGINALD RYVES. Eng. News-Rec., 103: 362–3, September 5, 1929. Brief illustrated outline of Lloyd dam at Bhatgar, in Bombay Deccan, completed last year, which is one of series of great projects of large irrigation scheme. Structure, which is of masonry, has volume of 21,500,000 cubic feet, is 5,333 feet long, 190 feet high above lowest foundations, and will create reservoir of 24,198-million-cubic-foot capacity with water depth of 143 feet over lowest sluiceways. Area of lake is 14½ square miles and average depth is 60 feet.—*R. E. Thompson.*

Concrete Sewer Under 83-Foot Head at Venice, California. Eng. News-Rec., 103: 365–7, September 5, 1929. Illustrated description of construction of 8100 feet of pressure sewer, comprising part of extension of Los Angeles sewer system to serve newly annexed territory in Venice. Bids for cast iron and reinforced spun concrete pipe were, respectively, $131,853 and $104,407. The spun concrete pipe was therefore adopted. Two sections of pipe selected at random from each lot of 25 were subjected to hydrostatic pressure of 45 pounds per square inch at the casting yard. Specifications required that field test of the completed line be made with open ends bulkheaded so that line could be filled with water and allowed to stand 4 days. Thereafter a 24-hour continuous test was to be made with an internal pressure amounting to 125 percent of the maximum service pressure to be employed (36 pounds per square inch), leakage under these conditions not to exceed 200 gallons per inch diameter of pipe per mile per 24 hours, loss being measured by metering the water pumped into the line to maintain the required pressure. The total leakage amounted to only 20 percent of that specified. The pipe was made of 1:1:2 mix, with no aggregate larger than ¾-inch. Specifications called for minimum crushing strength of 24,000 pounds per 4-foot length of 36-inch pipe. Pipe was supported in concrete cradle as much of the territory traversed was swampland or tide flats. Base was reinforced transversely to prevent cracking. Depressions were provided for the pipe bells. The joints were caulked tight with strands of hemp impregnated with thin cement grout.—*R. E. Thompson.*

Subway Excavation by Tunneling Methods Using Mining Shovels. Cont. Rec. and Eng. Rev., 43: 1225, October 9, 1929. Brief outline of methods being employed on Route 107, Section 12, of New York 18-foot subway which passes beneath Newtown Creek, 87 feet below the surface, with particular reference to the employment of 3 electric mining shovels. Although weighing only about 13 tons, machine has crowding force of about 12 tons. Rock is being loaded into 2½-yard cars running on a narrow gauge track. These machines averaged between 60 and 70 such loads per 8-hour shift, working 24 hours each. Material being handled is Fordham gneiss.—*R. E. Thompson.*

Cast Iron, Steel, or Reinforced Concrete. CALEB MILLS SAVILLE. Eng. News-Rec., 103: 400–4, September 12, 1929. Detailed discussion of choice of material for large water pipe, particularly for mains from source of supply to distribution system, from standpoint of experience at Hartford, Connecticut. Details are given of specifications on which bids were asked on 3 pipe lines constructed in 1913, 1927, and 1929, contracts being awarded for cast iron,

reinforced concrete, and Lock-Joint pipe (steel cylinder covered internally and externally with concrete), respectively. Author points out that in 1913 it was good practice to compare 42-inch pit-cast-iron pipe with 44-inch riveted steel, putting a differential of 15 percent in favor of cast iron over steel for durability, while in 1929, 42-inch cement-mortar-lined pipe of cast iron or steel, together with pipe of reinforced concrete, were compared with 48-inch pit-cast pipe with ordinary tar lining. Waters from eastern slope of Talcott Mountain of Hartford system are particularly corrosive, necessitating special emphasis on this factor. Steel is much more susceptible to corrosion than cast iron. Reduction in carrying capacity due to tuberculation has been found to be as much as 28 percent in 15 years. It has been found practically impossible to keep shop coating on outside of steel pipe intact during construction. Coating applied to abrasions in the field are seldom or never as permanent as that applied in the shop. Details are given of "Pabco Soil Proof Wrap," which has been in successful use on steel pipe in the west for some years and which was specified for steel pipe for 1929 line.—*R. E. Thompson.*

British Water Pipe Practice and Experience. M. N. BAKER. Eng. News-Rec., 103 : 405–7, September 12, 1929. An outline of practice in Great Britain. For distribution mains, cast iron pipe with bell-and-spigot joints is usually employed. Steel seems to be most general substitute for cast iron with spun or centrifugally-cast concrete second. Bituminous coating, spun in rather than applied by dipping, is generally used for steel. For external protection, use of "Hessian" soaked in bituminous material is quite general. Lead, run in hot, is used almost wholly for cast iron pipe joints. Circumferential joints in steel water mains appear to be most commonly of the hub-and-socket type, lead caulked. Breaks in cast iron mains appear to be less common than in America. Breaks which have occurred have been due to corrosion or defective foundations. Bedding in concrete or concrete bracing at bends are among the remedies employed. Outlines are given of practice and experience in several of the larger cities. In clayey London soils, cast iron pipe sometimes becomes so soft (graphitic) that it can be cut with a knife. This was formerly attributed to electric currents due to irregularities in the iron, but recent studies seem to show that softening is caused by "clay bacteria." In clayey soils cast iron pipe is now surrounded with gravel. Experiences with cast iron in the original duplicate conduits of the Vyrnwy aqueduct to Liverpool have led to adoption of steel for the third line. Spun-in bituminous lining is being employed, cement lining being attacked by the water. Cleaning of conduits is necessary from time to time to remove incrustation. Whalebone brushes, which are driven by water pressure, are used for this purpose and are described and illustrated. Much trouble has been experienced from both breaks and incrustation in the cast iron portion of the Elan aqueduct of Birmingham. This had led to the installation of supplementary steel pipe lined with spun-in cement mortar at some of the siphons. Breaks have been due to foundation conditions. Portions of the line which have been subsequently bedded in concrete have given no further trouble. Interior of both the cast iron and the steel pipe in the original aqueduct are hand scraped from time to time to remove incrustation. It is pointed out that ball-and-socket joints in cast iron

pipe were first used in 1811 in Glasgow, being introduced by JAMES WATT.—*R. E. Thompson.*

Steel Used for Large Water Mains in New York City. W. W. BRUSH. Eng. News-Rec., 103: 408–9, September 12, 1929. Article deals only with mains of diameter in excess of 30 inches, as cast iron still holds the field without serious competition for smaller sizes. Steel pipes up to 72 inches in diameter have been laid, the plate thickness employed being ¼ inch. Cast iron has been abandoned for large mains owing to breaks, usually caused by external loading and foundation conditions. Author believes that steel mains should be cleaned and recoated in place every 15 to 25 years and that the life of such mains laid in soil such as found in New York should be more than 100 years, if not destroyed by electrolysis. The minimum diameter of pipe which can be reasonably cleaned and recoated in place in 30 inches. As instance of damage by electrolysis, it is noted that a 66-inch steal main laid in 1925 was so badly pitted last year that 25,040 pits had to be filled in by welding in a stretch of 723 feet, at a cost of $14,304. Contractor is allowed to choose the type of longitudinal joint, the electrically-welded type being the one usually offered. Specifications require that this joint must show virtually 100 percent of plate strength, break outside of the weld in the tensile test made on coupons cut from the finished pipe, and bend cold 180° around a 2¼-inch pin without sign of fracture. No allowance is made in specified diameters for different materials bid upon. Tuberculation of steel pipe will have more effect on the capacity than will the type of joints and no one can determine with certainty the actual capacity coefficient of a given pipe line when new or after 20 years. Single-riveted circumferential joints are employed but are not considered entirely satisfactory as they develop only 56 percent of strength of the plate at the part of the line which is most likely to break should settlement take place. Hot dip or hot bitumastic hand-applied enamel may be used at the option of the contractor, the minimum thickness specified being $\frac{1}{16}$ inch. Experience indicates that dip coating will last about 15 years before it should be renewed. —*R. E. Thompson.*

Temperature Variations Cause Trouble on Jerusalem Pipe Line. A. PALOTAN. Eng. News-Rec., 103: 415, September 12, 1929. Brief description of two experiences due to extreme temperature variations on 8-mile, 8-inch pipe line laid with Victaulic joints 3 years ago. Greater part of pipe line is exposed to air and sun and temperature may vary from 60° to 131°F., depending on the time of day. In one case pipe had increased in length due to increase in temperature during repair operations, necessitating cooling to effect replacement of section removed, and in another, the pipe had contracted, necessitating heating of section of line to enable connection to be made.—*R. E. Thompson.*

New Dam at Lac Seul in Northern Ontario Provides for 150,000-H.P.: Power Development. Cont. Rec. and Eng. Rev., 43: 323, March 27, 1929. A brief illustrated description of Lac Seul dam, which is of the typical stop log concrete gravity type, 590 feet in length overall, with maximum depth of 45 feet from sill to deck. The dam was constructed by the Ontario Government, by

arrangement with the Dominion Government, to further regulate the flow of the English River and the Manitoba reach of the Winnipeg River.—*R. E. Thompson.*

British Rainfall, Water Consumption, and Water Treatment. M. N. BAKER. Eng. News-Rec., 103: 497–500, September 26, 1929. Data are given on the recent drought and water shortage, water consumption, and water treatment in Great Britain. Tabulated summaries are given of consumption and treatment statistics. Of 633 cities, 121 have an average per capita consumption of less than 12 (U. S.) gallons per day, and only 5 and 8 have consumptions in excess of 100 and 85 gallons respectively. The consumption in 80 percent of the cities is between 12 and 47 gallons. Domestic consumers are rarely metered. Of 642 supplies tabulated, 241 are filtered and only 28 chlorinated. Sheffield's supply is obtained from a near-by storage reservoir on 4 streams. Right to divert 5 m.g.d. from the Don River was conditioned on providing an equal amount of compensation water. To meet this requirement, water is pumped from the river below the sewage works outlet, softened with lime and soda ash, coagulated and filtered, pumped to elevation of 350 feet through 11 miles of 26-inch pipe, and returned to the river just below the reservoir. The plant had not been operated for 4 or 5 years until the recent drought, water not having been required from this particular reservoir. This is believed to be the most elaborate plant in existence for treating compensation water.—*R. E. Thompson.*

Many Economic Problems Involved in Proposed Changes in Public Land Policy. KENDALL K. HOYT. Eng. News-Rec., 103: 427–9, September 12, 1929. Incidentally, it is pointed out that destruction of forage growths by overgrazing permits erosion, an indirect result being the silting of streams. Regulation of grazing on open ranges has often been advised, but never attempted.—*R. E. Thompson.*

Crawler Trailer with 9-Cubic Yard Body Used at Rockfill Dam. HECTOR KEESLING. Eng. News-Rec., 103: 616–7, October 17, 1929. Brief illustrated description of crawler trailers drawn by crawler tractors being employed for hauling the spillway excavation to the fill at the Salt Springs rockfill dam under construction by the Pacific Gas and Electric Co., on the Mokelumne River in California as part of power development.—*R. E. Thompson.*

Gravel-Wall Wells. Anon. Railway Eng. and Maint., 26: 8, 336–338, 1930. The Chicago, Rock Island, & Pacific R. R. solved a critical water supply problem at Shawnee, Okla., by installing 6 gravel wall wells, 42 inches diameter by 40 feet deep, after oil well development had polluted the Canadian River with salt water drainage to the extent of 288 grains per gallon which caused serious corrosion and foaming in locomotive boilers. The gravel strata underflow of small creek was located by 72 test holes. Six wells were installed at most favorable location and operated by vertical centrifugal pumps with remote control. Water contains 31 grains per gallon dissolved solids of which 24 grains are incrusting. Lime, soda ash, and sodium aluminate are used for

treatment in 18- x 28-foot steel upflow reaction tank with two 75 feet diameter by 16 feet deep concrete settling basins. Total capacity of wells and plant is 1.5 m.g.d. Results have been very satisfactory.—*R. C. Bardwell.*

Preventing Leaks in Pipes. J. A. RUSSELL and R. C. BARDWELL. Railway Eng. and Maint., 26: 8, 349–350, 1930. Advantage of good materials and workmanship is emphasized.—*R. C. Bardwell.*

West Mill Waterworks: Bishop Auckland, England, Urban District Council. Anon. Water and Water Eng., 32: 376, 155–159, April 19, 1930. This supply, providing for population of 15,000, is obtained primarily from gravel beds near the River Wear, which act as preliminary filters. Water from the collecting system enters suction well from which it is pumped to mechanical filters of Bell vertical pressure type handling 120 gallons per square foot per hour with alum dosage varying from 1 to 2 grains per gallon. From pressure filters the water, still under pressure, passes to three sand filters. Effluent from these is pumped to service reservoir. Pumping installations are briefly described and Crossley three-cylinder vertical compressor-less Diesel engine is discussed in considerable detail. Article closes with report on chemical and bacteriological characteristics of finished water.—*Arthur P. Miller.*

Mud-Flush Rotary Drilling for Water Wells. L. P. TIMMINS. Water and Water Eng., 32: 376, 161–163, April 19, 1930. Rig for this kind of drilling consists of steel derrick about 80 feet high and roller-bearing rotary table chain-driven from two-speed draw works which is in turn driven by single-cylinder steam engine. The mud fluid used is drawn from sump by powerful mud pumps and circulated to bottom of well through hollow drill stems and hole in the bit, returning up the drill hole carrying the drillings and then back to sump. Water wells are most often drilled either by the chilled shot coring method, or by the percussion method, using solid rods and a slip rope. Advantages of mud-flush method over these are: (1) increased speed of drilling, because there are no delays for baling, as with percussion method, nor for recovering cores; (2) less chance of well caving, because of plastering of its sides by the mud fluid; (3) more simple casing program; (4) well diameter kept constant for greater depths; and (5) fishing jobs less frequent and, when necessary, simpler.—*Arthur P. Miller.*

Borehole Pumping Plants for Municipal Waterworks: Details of Some Recent English Installations. Anon. Water and Water Eng., 32: 376, 169–171, April 19, 1930. In the past, steam-driven reciprocating pump was type generally used for raising water from deep wells or boreholes. Advantages of centrifugal borehole pump are now bringing it into more extensive use. Borehole may be of less diameter; pump house may be smaller and foundations lighter; but principal advantage is absence of valves and consequently of valve troubles. Application of borehole pump to well is discussed and article concludes with a few examples of satisfactory installations.—*Arthur P. Miller.*

Treatment of Industrial Waste Water. Anon. Water and Water Eng., 32: 376, 171–178, April 19, 1930. Industrial owners should recognize their two-fold obligation: first, to avoid pollution of streams, and second, to carry on their work economically. If treatment of factory wastes were regarded as essentially a factory liability, it would probably receive more consideration and attention. In studying waste treatment, the industrial process should be carefully examined to determine whether materials have been used to best advantage, whether yields are in accordance with theory, and whether all by-products which are usable have been extracted. Objects of industrial waste treatment are three-fold: (1) retention of suspended solids; (2) supression of acidity and of animal and vegetable poisons; (3) reduction to a reasonable strength of alkalinity and organic materials. In a very good discussion of the three usual processes, mechanical, chemical, and biological, four sub-divisions of last-named are made, into natural biological purification, purification by the soil, artificial biological purification, and activated sludge. In the second part are discussed the aims and processes in treating wastes from various trades. Trades discussed, with numerous subheadings under most of them, include those preparation of food, chemical industry, leather trade, grain and starch industry, metal trades, and the mining, paper and cardboard, sugar, and textile industries. Discussions are brief, but thorough. In concluding, author covers briefly legislation relative to stream pollution.—*Arthur P. Miller.*

Waterworks Construction. Part II. Pipe Laying. M. RATCLIFFE BARNETT. Water and Water Eng., 32: 376, 179–183, April 19, 1930. Reduction in capacity of water pipes has been much discussed. In years passed it has always been assumed to be due only to incrustation and corrosion; but there are other reasons for it. Incrustations are formed by the corrosion of iron pipes when the inner surface is not protected. No chemical treatment, not even neutralization of acid water, will altogether prevent the corrosion of iron pipes The author gives the coating clause which he uses in all pipe specifications. This coating has been found to be of good quality and of very satisfactory durability. Among non-corrosive pipes discussed are wood stave pipe which the writer concludes "would not be generally suitable in this country" (England), reinforced concrete pipe, and a cement-asbestos pipe now made under the name of "Everite." Another agent causing flow reduction is a black slimy growth investigated by Prof. J. CAMPBELL BROWN in connection with the Liverpool Waterworks aqueduct, which retards the flow of water without much reducing diameter of pipe. Concrete-lined or bitumen-lined pipes are just as subject to this slimy growth as are iron pipes. The only effective remedy is to have the water treated and filtered before it enters the pipe lines.—*Arthur P. Miller.*

Vitality of Certain Micro-Organisms Found in Water. Jour. Amer. Med. Assoc. 94: 730, March 8, 1930. This is an excerpt from letter from regular Paris correspondent. Mr. TRILLAT, Professor at the Institut Pasteur, in special studies of longevity of colon and typhoid bacilli in water, found that these organisms can persist in water in such an attenuated state that they

escape discovery in a bacteriological analysis. Under certain influences such bacteria may be awakened to new life and proliferate abundantly. In studying the general curves of morbidity, TRILLAT was able to verify the relations which exist between sudden barometric depressions, the augmentation of bacteria, and morbidity.—*J. H. O'Neill.*

Contamination of Water Supply with Proteus as a Cause of Epidemic of Proteus Infections. K. BECKMAN and R. HURTHLE. Deutsche medizinische Wochenschrift, Berlin, **55**: 1628, 1929. (Abstract, Jour. Amer. Med. Assoc., **93**: 1847, December 7, 1929.) Simultaneously with the contamination of the water supply by *Proteus vulgaris* numerous cases of proteus infections were observed. In the majority of patients the disease had the clinical aspects of a gastro-enteritis.—*J. H. O'Neill.*

Purification of Water by Electroosmosis. EDWARD BARTOW and R. H. JEBENS. Ind. Eng. Chen., **22**: 1020–1, 1930; cf. C. A., **22**: 470. Using apparatus designed and manufactured in Germany (C. A., **22**: 2017), purified water was obtained more or less equivalent to distilled water and at about the same cost. Water with residue of only 15 p.p.m. was obtained from raw water having 725 p.p.m. This residue was mainly silica. The apparatus therefore does not remove this inpurity. The work reported demonstrates the practicability of this method for water purification.—*Edward S. Hopkins* (*Courtesy Chem. Abst.*).

The Rapid Sand Filter. JOHN R. BAYLIS. Water Works and Sewerage, **77**: 7–10, January 1930. Filter beds are usually of standard construction, rectangular or round, of any size up to 1500 square feet of filtering surface, and with 24 to 30 inches of sand. The water is passed through at a rate of 2 to 3 gallons per square foot per minute. The beds are washed by forcing filtered water through them in the reverse direction to that of filtration at a rate sufficient to partially suspend the sand. When the sand is clean, there is little shrinkage during filtration; but where the sand grains have become coated with a gelatinous coating, some beds have a shrinkage of 5 percent or more by the time the loss of head reaches its maximum. This is responsible for cracks opening up in the surface and along the sidewalls of the filters. When coagulated matter begins to concentrate on the surface the bed becomes more resistant to the flow of water. At some points coagulated matter forced together on the surface of the bed checks the flow, while at others, breaks occur, flow is increased, and coagulated matter is taken farther into the bed. Where breaks occur, channels open up from the surface down into the sand for a short distance. Where considerable penetration has taken place, the rate of flow through the bed should not be increased suddenly. The writer favors a treatment prior to filtration that will give a coagulated turbidity of not more than five.—*C. C. Ruchhoft* (*Courtesy Chem. Abst.*).

Temporary Suspension Bridge Carries 36-inch Pipe Line over Landslide. C. A. LAUENSTEIN. Water Works and Sewerage, **77**: 17–18, January 1930. In order to prevent damage to a 36-inch pipe line by a landslide in California,

a 170-foot suspension bridge was built to span the slide. Work was begun within two days after slide started. Anchors were of concrete, having a total weight equal to twice the cable pull and an area to limit the earth pressure to 2000 pounds per square foot. Each cable anchor required about 13 cubic yards of concrete. Thirty-foot towers were selected. Eight strands of $\frac{7}{8}$-inch cable were used. When completed and loaded, cable stress was 52,000 pounds as against a safety strength of 60,000 pounds. The work was completed in 17 days at cost of $17,000.—*C. C. Ruchhoft.*

Water Supply for Los Angeles from High Sierras. WALTER A. GILLETTE. Water Works and Sewerage, 77: 21–22, January 1930. A large quantity of water can be diverted for use in the Metropolitan District of Los Angeles from six large river basins within the state of California. The Owens River Basin has drainage area of 524 square miles which produces a mean run-off of 278,000 acre-feet and would supply 1,405,000 people. Bishop Creek has 446 square miles area, producing 341,500 acre feet, and capable of supplying 2,340,000 people, and Owens Lake can serve 570,000 people. Mano Lake Basin can supply 70,000; East Walker River, 1,075,000; the West Fork of Walker River, 1,075,000; or a total of 2,850,000. These basins can be developed as the need arises. All six can be developed at an estimated cost of $100,000,000.—*C. C. Ruchhoft.*

The New Water Works of Cedar Rapids, Ia. H. T. RUDGAL. Water Works and Sewerage, 77: 1–5, January 1930. The Cedar Rapids water works consists of a low-lift pumping station and a 12-million-gallon daily capacity softening and filtration plant. The filters, clear well, suction channel, premixing tanks, and pumping station substructures are built of reinforced concrete. Masonry construction is used for the superstructure, with roofs supported on steel trusses and beams. The high-lift plant includes two 50 feet square mixing tanks, 22.5 feet deep, two clarifier tanks 100 feet square with depth of 16.5 feet at walls and 19.8 feet in center, and a carbonating tank 40 feet square and 16.5 feet deep. The excavation from the pumping station totaled 8500 cubic yards and for the tanks, 7400 cubic yards. All reinforced concrete was 1:2:4 mix, to which was added 3 pounds of Celite per bag of cement. The high-lift station structures required 4400 cubic yards of concrete and 418 tons of steel. The exterior walls of the pumping station are faced with brick and cast stone. Seventy-three tons of flange pipe and fittings and 362 tons of bell and spigot pipe are used in the installation.—*C. C. Ruchhoft* (*Courtesy Chem. Abst.*).

Cracks in Filter Beds. JOHN R. BAYLIS. Water Works and Sewerage, 77: 37, February, 1930. Cracks formed on the surface of filter beds and those along the side walls, which later may extend downwards to 60 or even 80 percent of the total depth of the sand, are caused by shrinkage and settlement of beds. The sand grains of a filter become coated with a material consisting, when dried, as to 30 percent of organic matter and as to the rest, of SiO_2, Al_2O_3, Fe_2O_3, and MnO_2. Surface cracks form at times when coagulation is only moderately strong, as in the spring and fall. When the top 2 or 3 inches of sand is more heavily coated with gelatinous material and is more loosely packed

than the rest of the bed, cracks form readily. Side cracks are more dangerous and may allow coagulated matter to pass the bed. Cracks lengthen filter runs, producing a larger filtering surface, and they cause clogging. Cracks may be eliminated or prevented from forming, by keeping the sand grains so clean that there will be practically no settlement of the bed as the loss of head increased. Surface wash is the most practical means of keeping the sand clean all the time.—*C. C. Ruchhoft* (*Courtesy Chem. Abst.*).

Los Angeles County to Store Excess Flood Water. Anon. Water Works and Sewerage, **77**: 49, February 1930. A total of 22½ billion gallons of water will be added daily during flood-water periods to Los Angeles County's naturally stored underground supply by the capture and infiltration of flood water that now runs into the ocean. The flood water will be taken by one main or several canals and conveyed into canals from where it will percolate. The water can also be stored in natural subterranean reservoirs and dammed in natural underground dykes.—*C. C. Ruchhoft.*

Coagulation Basins, Baldwin Filters, Cleveland. Anon. Water Works and Sewerage, **77**: 43, February 1930. The coagulation basins at Cleveland, O., have features that are departures from ordinary practice. A straight flow through the basins with no baffles has proved satisfactory. The floor slopes sharply for the purpose of reducing height of outside walls and facilitating cleaning of the basins. The basins have an average water depth of about 15 feet and are of reinforced concrete. Tables and figures give details of design. —*C. C. Ruchhoft.*

Aëration in Water Purification. MALCOLM PIRNIE. Water Works and Sewerage, **77**: 58, February 1930. To be non-active, waters having bicarbonate alkalinities of less than 100 p.p.m. must not contain more than 2 p.p.m. of free CO_2, resulting in a pH between 8.1 and 8.3. The CO_2 can be expelled by fine-spray aëration from clean waters down to a residual of from 2 to 4 p.p.m. The residual can be neutralized by adding hydrated lime at rate of from 0.1 to 0.5 grain per gallon. Operating results are given for West Palm Beach, Fla., Providence, R. I., Watertown, N. Y., and Danville, Va.—*C. C. Ruchhoft* (*Courtesy Chem. Abst.*).

Zeolite Water Softening for Municipal Purposes. H. B. CRANE. Water Works and Sewerage, **77**: 59, February 1930. In the operation of a zeolite water softening plant, raw water that needs no filtering is pumped through the softener directly into the distributing system. When the capacity of the zeolite has been utilized, the raw water pump is stopped and a brine solution applied to restore the softening properties of the zeolite. In a lime-zeolite plant, in which pressure shells are used as containers for filter sand and zeolite and a steel or wood tank for coagulation, the raw water is pumped to the top of the coagulating tank where lime is added to remove the carbonate. After settling, CO_2 is added to stabilize the water and prevent after-precipitation on sand or zeolite beds and in pipe lines. The high service pump then forces the water through the sand clarifier and also through the zeolite beds where re-

maining hardness is removed and thence to the distributing system. Where a definite hardness of water is required, suitable proportions of softened and partially softened water can be mixed. Descriptions of the zeolite softening plants at Springdale, Pa., and Mangum, Okla., are given.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

Filter Bed Clogging. JOHN R. BAYLIS. Water Works and Sewerage, 77: 73–77, March 1930. Clogged places in filter beds are areas of sand bound so firmly together that they are not broken up during washing. When the clogged masses are small and distributed throughout the sand bed they have no significance in filtration, but they are breeding places for bacteria. When they form into large masses and settle to the bottom of the filter they cause trouble. The major factor in clogging is the character of the water. Silicious material, clay in suspension, and water that causes the formation of gelatinous coatings on sand grains give the most clogging trouble. The first visible evidence of clogging is slightly raised places in the sand surface and clogged places along the sidewalls where cracks form. Clogging also occurs in the center of the bed beginning with small masses which form on or near the surface which, later on, accumulate and ultimately settle at certain points in quantities sufficient to start clogged areas. The design and operation of a filtration plant are also factors in determining the condition of filter beds.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

A Recent Development in Lime Water Softening. CHARLES P. HOOVER. Water Works and Sewerage, 77: 103–4, March 1930. This process relates to a method of reducing carbonate hardness of water by lime softening to approximately the solubility limit of calcium carbonate and magnesium hydroxide. Sufficient lime (CaO or $Ca(OH)_2$) is added to the water to combine with the free and half-bound CO_2 and to precipitate the magnesium hydroxide. About 25 to 50 p.p.m. of lime in excess is required to convert the magnesium salts to the hydroxide. This leaves excess lime in the water. After partially settling, the excess lime is neutralized with CO_2, then calcium carbonate, water softening sludge, or other inert, finely divided material, is mixed with the water, after which it is clarified by settling or filtration. Comparative processes and operating data are given.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

Experiences of a Filter Plant Trouble Shooter. L. E. WICKERSHAM. Water Works and Sewerage, 77: 93–94, March 1930. The preparation of water for filtration is a determining factor in successful filtration plant operation. The elements of time, temperature, hydrogen-ion concentration, and agitation determine effective coagulation and settling of water. After proper coagulation the water must be agitated as little as possible to allow a greater degree of settling. The "jar test" should be made to determine the amount of chemicals necessary for efficient treatment of water and time required for coagulation and subsidence. Red water trouble caused by high CO_2 content can best be met by aëration and eventual alkali treatment in clear-water basin. Where algae are present causing odor and taste and increased alum dosage, chlorination and washing of filters will rid the water of the organisms. Constant

care of the filters must be taken to prevent mud balls and cracks.—*C. C. Ruchhoft* (*Courtesy Chem. Abst.*).

Denver Installs New Water Conduit. J. C. COYLE. Water Works and Sewerage, **77**: 65, March 1930. The latest improvement in Denver's 23 million dollar water system is an 8-mile installation of 60-inch reinforced concrete conduit. Difficulties encountered were due to swamps, hard rock, two river crossings, and frozen ground. The width of the trench was usually 8 feet and the depth, 9½ to 15 feet. In crossing the river a cofferdam was built. The total cost of the line is $1,300,000.—*C. C. Ruchhoft.*

Sanitary Defects of Ground Waters. R. E. LAWRENCE. Water Works and Sewerage, **77**: 86–88, March 1930. Statistics show that both surface and underground waters are subject at any time to risk of contamination resulting from sanitary defects. Contamination can be prevented by careful construction and operation. Records show that of the ground water supplies in Kansas 36 percent met the U. S. P. H. S. standard.—*C. C. Ruchhoft.*

Damages Awarded for Death from Water-Borne Typhoid Fever. Anon. Water Works and Sewerage, **77**: 88, March 1930. In a decision handed down by the Indiana Appellate Court, damages were awarded for a death from typhoid fever. The administrator of the decedent brought action against the Pennsylvania R.R. and the City of Fort Wayne. A connection existed between the city's potable water supply and the railroad's supply from the river. In the summer of 1923 there were many cases of typhoid and in November 1923 there were 140 cases in the vicinity of the by-pass valve, and the plantiff's decedent was one of the persons afflicted. The claim was that the city water had been contaminated by the river water and that the person's death was due to drinking the city water. A jury trial resulted in a verdict and judgment against both defendants and the judgment was affirmed by the Appellate Court.—*C. C. Ruchhoft.*

The Water Works of the City of Mexico. EDUARDO MELINA, ANTONIO CORNEJO, and ALFONSO VILLA ACOSTA. Southwest Water Works Journal, June, 1930, pp. 11–17. July 18, 1325 was, according to SIGUENZA, date of foundation on plateau of Valley of Mexico, 7000 feet above sea level and enclosed on all sides by mountains, of Mexico City by the Aztecs, whose kingdom ended August 13, 1521. Chapultepec Springs, then outside, but now within city, were chief source of supply during this period; although in 1499 6-mile aqueduct was constructed to bring Xochimilcho water to the city, use of which was soon abandoned by reason of a disastrous flood and never resumed. Thereafter until present century four groups of springs to westward of city were main supply. Their yield, reckoned at 49 gals. per capita daily, was supplemented by 1070 private wells. Both quality of water and condition of distribution system left much to be desired when in 1902 the municipal authorities initiated investigations which resulted in choice of Xochimilcho Springs for present supply. Water is of high purity, being derived from an underground reservoir of considerable extent in the southwest of the Valley fed from the

surrounding mountains. Origin of this formation is described. Present Lake Xochimilcho overlies part of the reservoir, but pressure in latter is greater, so that contamination is not to be feared. Four pumping stations were chosen at such distances apart that underground water level would not be unduly lowered. The concrete aqueduct, 29 miles long, is at first of circular section, 54 inches in diameter; it enlarges after four miles to an ovoid section 6 feet 4 inches high by 6 feet 8 inches wide. Stations are provided at 1000 feet intervals for pressure relief, ventilation, and emergency closing gates. Main pumping station directs the water from aqueduct into the distribution system and the four 12½-million-gallon concrete-covered equalizing reservoirs. Each of three existing and fourth almost complete pumping units discharges 20,000 gallons per minute at 65 pounds pressure and is coupled to a synchronous 1400-kw., 3000-volt, A.C. motor. The 100-h.p. induction-wound starting motors work on same current. Suction and discharge are 36-inch; each of two 48-inch headers takes combined discharge of two pumps and carries 48-inch Venturi meter registering discharge. Current is delivered at 20,000 volts and stepped down to 3000. The arrangements of the equalizing reservoirs are fully described and leading particulars given of distribution system, including hydrants and meters. Growth of city has been phenomenal; population is now 1,000,000 and present supply of 63 m.g.d. delivered at city (out of 70.3 m.g.d. collected; difference is partly delivered to certain villages along line of aqueduct and partly lost by leakage) must be increased to at least 100 m.g.d. in near future. Owing to evidence of intermittent contamination, believed to originate mainly in a section of the aqueduct traversing difficult terrain, chlorination (Wallace and Tiernan) was inaugurated in 1929. One illustration depicts authors with V. M. EHLERS of Austin, Texas, making a chlorine determination. Article is well illustrated with figures and photographs and should be of particular interest to those attending A. P. H. A. Fort Worth Convention and Mexico City Tour.—*Frank Hannan.*

Court Decisions Relating to Public Health. Liability of City for Act of Health Officer. Public Health Reports, 45: 8, p. 407–408, February 21, 1930. (Ohio Supreme Court; City of Salem *v.* Harding, 169 N. E. 457; decided October 23, 1929.) The water and sewer gravity lines of city of Salem were laid parallel, a few feet apart. Certain property owners having failed to disconnect from the sewer as ordered by the city, the health officer caused the outlet to be plugged. It was claimed that the resulting backed-up sewage escaped through uncemented joints and found its way into the water main, causing a case of typhoid. Plaintiff brought action in trial court against the city to recover damages. A verdict in favor of the city was appealed to the court of appeals and judgment reversed. This was sustained by supreme court. A point also determined was that city was not a guarantor of the purity and wholesomeness of its water supply. **Protection of Public Water Supply from Contamination by Unlawful Bathing.** Public Health Reports, 45: 28, 1623-1625, July 11, 1930. (Connecticut Supreme Court of Errors; Harvey Realty Co. *v.* Borough of Wallingford *et al.*, 150 A. 60; decided April 17, 1930. "The plaintiff corporation owned land upon which was a small pond. Water from this pond flowed through a brook to the reservoir and pumping station

of the borough of Wallingford. The plaintiff set apart, for use by the public as a park, about 150 feet of land all around the pond. The land back of this strip was divided into lots for sale." The plaintiff invited the riparian and non-riparian lot-owners to bathe in the pond. The borough brought plaintiff's attention to the statute prohibiting this. The plaintiff brought action for an injunction to restrain the defendants from interfering with sale of its land and for damages. "The borough filed a counterclaim, asking for an injunction against plaintiff's use of its premises as a pleasure resort or as a rendezvous for swimming, boating, or fishing, or in a way that would render the waters unfit for water-supply purposes. The trial court found against the plaintiff in the action brought by it and in favor of the plaintiff on the counterclaim."—*R. E. Noble.*

NEW BOOKS

Das Berliner Heft. Kleine Mitteilungen des Vereins für Wasser-, Boden- und Lufthygiene, **30**: 9, 280 pp., 1930. This covers the second meeting of the association for water-, earth- and air-hygiene, held in Berlin on June 13–15, 1929, with an attendance of 427, which was in response to the successful attendance at the 1927 gathering in Essen. After the opening address of greeting by Dr. EICHHOFF, Mayor of Dortmund and chairman of the association and replies of welcome by Mayor Böss of Berlin and by other city officials, the mornings were devoted to the reading of papers, the afternoons to inspection trips, and the evenings to recreation. On the first (Thursday) evening the visitors were treated to a "Bierabend" by Dr. HIRTSIEFER, Secretary of State; on the second evening to similar festivities as guests of the city, and on Saturday noon to a closing banquet given by the Berlin Waterworks. The following papers were read at the morning sessions: **Water Collection, Purification and Use.** K. KÜHNE, director of the Berlin Water Company. When the per capita utilization of water was 100 liters per day, with peak of 130 liters, many cities experienced no difficulty in obtaining an adequate supply. Now the mean consumption has increased by 40 per cent and the peak load by 70 per cent, and it becomes increasingly difficult to provide for unexpected severity of climate such as the winter of 1928 which was the coldest in 100 years or in the 73 years' life of the Berlin Waterworks. The case of Wien, too, is cited, in which all the lakes froze so deeply that an adequate supply could not be maintained. The increased consumption is attributed to growth in the central heating units and to developemnt in other uses such as auto washing: the 60,000 autos in Berlin, each needing from 0.2 to 0.3 cubic meters of water for daily washing, require 15,000 cubic meters (4 m.g.d.). Among the possible remedies are more public baths: in these the warm water is sold directly to the people instead of to a landlord, and paying for the water impresses conservation on the individual; also in flushing toilets 10 liters of water are used, only a fraction of which is needed, and mechanical devices for reducing this waste are under consideration. Curtailment of hot water cannot be considered; its use is a measure of culture, and the total water consumption will probably reach 250 liters per capita. More than this is needless and wasteful, and the American consumption of 500 liters is not to be regarded as necessary, but as

an expression of abundant supplies. For drinking, the consumer desires a cool, bacteria-free, crystal clear ground water of medium hardness. Industries and cooking require soft water; e.g., modern high-pressure boilers require not over 2° hardness. Despite these requirements, reliability of quantity governs the choice of supply, since any reasonably clean water can now be rendered suitable for human used. Complete dependence can no longer be placed on ground water alone, and an increasing percentage of surface supplies is coming into use. Treatment processes fall into two main groups, the one for drinking and the other for industrial purposes. Drinking water must be free from suspended matter; soil or clay turbidity usually requires alum or lime coagulation. Iron and manganese are not directly injurious to health, but give turbidity and disagreeable tastes on oxidation; they are detrimental for laundries, photographic industry, paper manufacture, etc. Oxidation preceding filtration removes iron; manganese precipitates more slowly. The early treatment of surface supplies depended upon slow sand filters with a filtration velocity of 100 mm. per hour; the use of ground waters with iron and manganese removal led to the adoption of coarser sand with velocities of from 0.8 to 1.0 meter per hour, the action being mechanical and not biological; the most recent rapid sand filters have velocities of from 3 to 10 meters per hour. This last type was made possible by the widespread adoption during the last 15 years of liquid chlorine sterilization. Active charcoal is coming into use for taste removal, but the filters are costly to install and to regenerate. For industrial waters, softening and degassing are the main treatments. Softening is accomplished by the lime-soda, permutit, or calcium carbide process. The new electro-endosmose process may have promise, if it can be applied to large volumes of water. **Problems in Water Hygiene.** B. BÜRGER, director, bacteriological division of the Landesanstalt. Of first importance is the elimination of pathogens, especially the intestinal forms such as typhoid, the paratypnoids, Asiatic cholera, etc. Even animal excreta may carry human diseases, including the eggs of parasitic worms. Chemical poisons, such as arsenic and lead, must be absent. A separate supply may be provided for industrial use if all possible cross-connections are avoided; it is safer to chlorinate the industrial water very heavily and to use conspicuously painted pipelines for it. The care of the water-shed is discussed briefly with reference to fencing to exclude animals and planting with trees to decrease soil erosion. Filtration in any form does not completely remove bacteria, and disinfection is necessary especially for intermittently contaminated, surface water. Chlorine, ozone, or ultraviolet light may be used; whether the oligodynamic action of metals like copper and silver (such as KRAUSE'S "Katadyn") will be useful, is still an open question. The chlorinator should be equipped with an alarm signal. After disinfection, the total count should not exceed 50 per cc., and usually not over 20; *coli* should never be found in 200 cc. portions, and rarely in 0.5 and 1.0 liter quantities. At the Landesanstalt attempts are being made to develop a test showing any members of the group, rather than isolation of a well-defined *B. coli commune*; e.g. indol formation gives a presumptive test for the colon group. But even more important than the method, or the kind of organism, is the use of sufficiently large samples. The suggestion having been made to run the indol test on 0.1, or 1.0, cc. portions of samples from over

100 water works using ground or spring supplies, only one, and that only a single time, showed positive on 0.1 cc. 1.0 cc. portions were positive only for 1 city, and for that city only in 6 out of 58 samples, though the indol test is positive twice as often as *coli* isolation. The experience at the Anstalt has been that city supplies are positive, now and then in 200, 100, or 50 cc. portions; sometimes in 20 cc., rarely in 10 cc., and almost never in 1 cc. Waters with *coli* in 1.0 or 0.1 cc. can not be regarded as suitable for public supplies without treatment. This does not agree with some recent opinions to effect that the purest spring or well water will occasionally show *coli*, but that this showing is unimportant and to be regarded with suspicion only when frequent and considerable (at least in 1.0 cc.) *coli* appear. However, the Anstalt favors the Italian standard of no *coli* in at least 250 cc. of sample. For waterworks control, the French practice is 12 chemical and bacteriological samples a year for towns of 20,000 or more; for towns of from 5,000 to 20,000, samples 4 times a year, including the baths, bathing beaches, and tourist camps. A sanitary survey should be made at least once a year. The German practice on treatment plants is daily total count and *coli*, with careful supervision of chlorination. An official inspection is made at least once yearly, or for small plants, every 3 years. All of the staff are examined for typhoid carriers, especially after a known case of the disease when the subject is examined every 3 months for at least 3 years. **Corrosion and Water Purification Questions in the Light of the Newer Electrical Investigations.** L. W. HAASE of the scientific staff of the Landesanstalt. At the present day, a treated water after standing all night in a lead pipe rarely shows over 0.1 p.p.m. of lead; this is insignificant, though from 0.2 to 0.3 p.p.m. can cause poisoning. Lead solubility is due not merely to the softness of a water, but requires also oxygen and aggressive CO_2; deep well supplies, free from oxygen, may contain abundant aggressive CO_2 and yet show no solvent action on lead. However, soft waters, and even alkaline waters, containing high aggressive CO_2, but no oxygen, can dissolve iron mains (giving ferrous hydroxide and bicarbonate) and injure the water for drinking and laundry purposes. Such waters require about 5 p.p.m. of oxygen. Abundant oxygen leads to "red water," the ferrous bicarbonate precipitating as ferric hydroxide and the liberated CO_2 attacking more iron. The composition of the water should be such that protective coatings are formed in the mains; these may be natural (homogenetic) coatings of deposited ferric hydroxide, or artificial (heterogenetic) coatings of deposited carbonates. The latter are formed from waters high in carbonate hardness; a high non-carbonate hardness may be just as corrosive as a soft water. Continued use of high carbonate waters leads to incrustation and loss of capacity of the mains; this is particularly true of waters lacking aggressive CO_2, since in presence of oxygen the equilibria are shifted. Thus hardness alone does not tell the whole story, though for a first approximation we may accept KLUT'S value of 7° (125 p.p.m.) of carbonate hardness as necessary to give a protective coating. The ability of a water to form such a coating is now determined in the laboratory by (*a*) corrosion tests and (*b*) direct electrical measurement of the deposit. The former require large amounts of water and long times, especially if attempts are made to duplicate actual conditions. Accordingly the velocity of incrustation is often more conveniently determined by flowing the water between noble-

metal electrodes and measuring the voltage (field equipment illustrated). Water treated with alum or iron sulfate contains more permanent hardness and free CO_2 and hence is more corrosive. Saturating the water in a marble filter permits an adjustment of the equilibria so that its action on the pipe will shift these equilibria just enough to give protective coatings. **Practical Drainage and Sewage Treatment Questions of Municipalities.** F. LANGBEIN, director of the Berlin city drainage. Some of the engineering aspects of sewer construction are discussed, particularly with reference to interceptors and outfalls. Adequate ventilation, artificial if necessary, should be provided to prevent accumulations of explosive gas; safety lamps and enclosed electric switches are necessary, together with rigorous ordinances regarding discharge of benzol, or other inflammable liquids, into sewers. **Treatment and Disposal of Effluents from the Lignite Industry.** H. STOOFF, scientific member of the Landesanstalt. Of 166.2 million tons of lignite produced in 1928, 50 percent was briquetted and from 2 to 3 percent used for tar, coke, gas, oil, or bitumen manufacture. The principal wastes are (1) from the mining operations: ground water, hydraulic operations, wash water, etc., carrying coal dust, clay, and sand, and sometimes acid; (2) from coal washing; (3) from the miner's bathrooms, and (4) from briquetting. Electrical precipitation of atmospheric dust is replacing water sprays, because the former recovers a usable product. Discarded strip mines, stone quarries, or artificial tanks, are used for sedimentation; the period must be 6 hours, and should be from 1 to 2 days; scum baffles catch the floating dust. The settled sludge has a water content of 80 percent; this is reduced to from 40 to 60 percent by relatively costly filter pressing, the water being returned to the process and the cake burned under the boilers, or added to the briquette output. Briquette water requires precipitation of the colloids by iron or aluminum salts, $CaCO_3$, $MgCl_2$, lime-water, etc. Some use is made of the coagulating effect of ferruginous mine water, or coke-quenching water, on the briquette wastes. For coal distillation, the old, externally-heated, rotary kilns operating at about 500° handle about 1.5 million tons annually; this process produces from 1 to 3 cubic meters of water per oven, each handling from 2 to 5 tons of coal per day. The raw coal contains from 50 to 60 percent of water; air-dried coal, from 12 to 18 percent; while an additional 9 to 12 percent results from the chemical degradation. The waste water contains materials very toxic to stream life and very difficult to remove. The amount of waste can be lessened by air-drying the coal, but only 30 of the 995 ovens follow this practice. In the wastes are ammonia, pyridine bases, sulfur compounds, and cresols (instead of phenols as in anthracite). For fish life, 5 to 10 p.p.m. of phenol, 10 to 15 of cresol, less than 2 of cyanides, 4 to 10 of ammonia, 10 to 20 of hydrogen sulfide and about 100 of benzol are toxic; 1 gram quantities of thiocyanates, ammonium salts, or pyridine, in a liter are toxic. Phenol tastes in water or in fish can be given by 1 p.p.m. or less; the chlorophenol taste results with less than 0.1 p.p.m. The new ovens, using both internal and external heating, now number 29 and handle about one million tons annually; they use air-dried coal, or briquettes, at the rate of 100 tons or more a day, each yielding 10 to 15 cubic meters of waste water. The wastes from gas production depend upon the process used. Those from coke quenching amount to from 0.3 to 1.0 cubic meter per ton of coked coal. Other industries, such

as hydrogenation, mineral oil distillation, and bitumen manufacture, contribute some wastes. From liquors containing 2 percent or more of phenol, 90 percent can be removed by extraction with benzol, toluol, or tar-oil fractions. Phenol liquors (under 0.01 percent phenol) from the old distillation process can be added to domestic sewage to the extent of 20 to 25 percent without injuring its biological purification. The most effective methods of phenol removal are the various physicochemical processes: air oxidation, absorption on ashes, etc. (bibliography given, but no details). Phenol up to 10 p.p.m. is quickly destroyed biologically in natural waters, but wastes entering a river should not exceed this value after dilution; if the river contains fish, or is to be used as a water supply, higher dilutions are necessary. The other constituents will also require regulations when their detrimental effects are better understood. **Present Questions in Street Cleaning and Dust Prevention.** G. ERDMANN, director of street cleaning for Berlin. Improved motor-driven brushes and water sprays are described. Dust prevention is usually by hygroscopic materials such as liquors from potash or ammonia-soda industries which are rich in the chlorides of magnesium and calcium; these become too dry in the hot summer and too wet in the rainy season, but their low freezing point prevents ice formation. Petroleum oils have been used successfully on macadam streets. The street sweepings have been applied with success to 13,500 acres of pasture at the daily rate of 20 car-loads of 20,000 kg. each; the material is spread in a 4 cm. layer and ploughed under to a depth of 30 cm., thereby saving half the purchased fertilizer. **Technical Aspects of Cadaver Collection and Disposal, Including Confiscated Animals.** A. SCHILLING, technical director of the Berlin slaughter-house. The total annual contribution of the Berlin district is: 730 horses, 32 colts, 4 donkeys, 115 cows, 165 calves, 456 pigs, 37 sheep, 23 goats, 100 deer, 10,884 chickens, 17,690 dogs and 11,775 cats. Types of collecting wagons are illustrated. Machinery is shown for sterilization at 5 atmospheres absolute (152°C.), to kill anthrax spores, and for extraction, etc. The three main products are fat, bone meal, and animal glue. **Experiences and Problems in Bath Sanitation.** W. v. DRIGALSKI of the Berlin Board of Health. The prevalence of indoor pools in Germany is attributal to the short warm season. The pool room should not be excessively large, but should be thoroughly lighted so that the pool contains no dark corners. The building walls should be white with but little dark decorative design; the pool walls also are white, though dark guide lines are permitted on the bottom. No disrobing should be permitted in the pool room; the lockers and showers are so arranged that the pool can be entered only through them. Photographs and floor plans are given. **Does Experience Justify the Expenditure for Pest and Vermin Extermination?** E. SELIGMANN and E. NEUMARK of the Berlin Hygienic-bacteriological Institute. Under consideration here are rats, mice, flies, mosquitos, fleas, bedbugs, lice, mites, ants, etc. The (annual?) damage done by rats alone has been estimated at 10 million Krone in Denmark, 15 million pounds in England and $200,000,000 in America. Meal-moths, by clogging milling machinery, have caused serious industrial shut-downs. Small yellow ants sometimes overrun hospitals, annoying the patients, destroying foodstuffs and sterile dressings, and polluting surgical appliances, bacteriological media, etc. Some household pests can carry disease, notably the intesti-

nal diseases, organisms of suppuration, T.B., worm eggs, malaria, yellow fever, plague, trichina, Weil's jaundice, probably anthrax, etc. Eradication is a function of the Hygienic-bacteriological Institute of the Central Sanitary Board; they operate (1) by giving advice and instruction in rat-catching, fumigating, etc., testing out commercial materials and devices, and public education through the press and journals; and (2) by coöperating with the police in organizing and equipping occasional clean-up squads. Eradication costs are low in summer but high in winter on account of the labor, materials, sprays, etc., required. For 1926–27 the expenditures were 120,000 RM and the receipts 42,000 RM (from the 1 RM tax per treated property). In 1927–28 the cost was 156,000 RM, of which 50,000 RM were collected; in other words, 2 to 2.5 Pf. per capita. Other examples of expenditures are: Breslau, 20,000 RM in a year for mosquito control; Leipzig, 35,000 for rat extermination; Magdeburg, 10,000 for rat killing and 5000 for mosquito control.—*S. L. Neave, Courtesy of Arthur M. Buswell.*

Proceedings of Eighth Annual Water Works School. Volume No. 2. February 12, 13, and 14, 1930, University of Kansas, Lawrence, Kansas, 125 pp. The **Advantages and Disadvantages of Combined Operation of Water and Light Plants.** J. W. DONOVAN, 11–17. Combined operation of water and electric light plants of Kansas City, Kansas, results in material economy and increased efficiencies through the use of the pumping plant and both condensing and municipal water supply, use of electrically-powered pump equipment at low cost, reduced overhead and less superintendence, operating labor, and administration and accounting forces. **A Recent Development in Lime Water Softening.** CHARLES P. HOOVER. 18–22. Discussion of benefits of excess lime treatment combined with return sludge in softening to limits lower than ordinarilly practiced. **Determining Factors in Municipal Water Softening with Special References to the Water Supplies of Kansas.** H. B. CRANE. 23–33. Discussion of the relative advantages and disadvantages of softening by means of lime and soda ash, zeolites, and a combination of these two methods. Brief descriptions are given of zeolite softening at Springdale, Pa., and at Mangum, Okla. **Iola Filtration and Water Softening Plant.** M. P. HATCHER. 34–36. New plants provided for treatment of raw water with lime, mixing through under-and-over baffled tanks, reaction in tanks employing the spiral inward flow principle, settling in existing basins providing a retention period of about 3 days, aëration through Sacramento spray nozzles, carbonation, treatment with alum, and filtration. **Increasing the Bacterial Efficiency of Water Purification Plants.** H. W. STREETER. 37–44. Author's summary: "In this brief discussion of the various measures available for increasing the efficiency of water purification processes, attention has been confined to the comparatively few of these measures falling within the range of systematic observations and experiments made by the Public Health Service which appeared to have outstanding merit, namely, double-stage coagulation-sedimentation and raw water prechlorination. Each one of these measures has given definite evidence of effecting a substantial and consistent increase in the efficiency of rapid sand water purification systems. In this connection, however, the use of improved mixing and reaction devices for coagulation, the prolongation of sedimentation

periods, and the addition of larger amounts of coagulant, afford possibilities for increasing bacterial efficiency such as well might be worthy of consideration by plant designers and operators having in hand the problem of meeting serious encroachments in raw water pollution. In general it seems that efforts to solve this problem in the near future, wherever it may appear, are more likely to consist in the tightening and extension of existing defenses than in the development of any revolutionary changes in current methods of water purification. The evidence that these safeguards are capable of such improvements is reassuring, both to water works officials and to others who are responsible for preventing a recurrence of the huge toll at one time exacted by water-borne diseases in this country, now happily and, may it be hoped, permanently erased from the list of our public health liabilities." **Ferric Coagulants.** L. L. HEDGEPETH. 47–50. Chlorinated copperas has been used for water coagulation at Elizabeth City, North Carolina, since 1928. Cost of chemicals for one year has been as follows:

1929	COPPERAS	LIME	ALUM	CHLORINE	TOTAL	CHEMICALS PER MILLION GALLONS	AVERAGE COLOR IN P.P.M.
January	$160.00	$60.75	$100.00	$101.25	$422.32	*$12.42*	300
February	136.00	55.87	138.00	108.00	437.87	*13.50*	300
March	150.00	63.75	165.00	81.00	459.75	*13.37*	325
April	161.00	63.75	174.00	108.00	506.75	*15.10*	500
May	148.00	56.25	156.00	87.75	448.00	*12.68*	310
June	102.00	39.38	156.00	74.25	371.63	*11.40*	245
July	120.00	45.75	150.00	87.75	403.50	*11.40*	500
August	148.00	58.88	171.00	114.00	492.63	*13.45*	400
September	148.00	50.62	159.00	114.75	472.37	*13.15*	350
October	131.00	66.37	168.00	101.25	466.62	*13.90*	300
November	128.00	65.25	147.00	94.50	434.75	*13.35*	240
December	136.00	66.00	129.00	94.50	425.50	*11.85*	225
TOTAL	$1,668.00	$692.62	$1,813.00	$1,167.00	$5,341.69	*$155.57*	
Average	139.00	57.51	151.82	97.25	445.15	*12.97*	332

Author concludes that "while ferric coagulants have remarkable coagulating powers, they are not 'cure-alls' for water work, neither are they necessarily adaptable in most cases." **Maintaining Chemical Balance to Resist Corrosion.** CHARLES P. HOOVER. 51–54. Benefits of $CaCO_3$ film in preventing corrosion of pipe walls are explained. The formation of this film can be controlled through adjustment of $CaCO_3$ in solution. **Cold Weather Troubles in the Operation of the Water Purification Plant, Lawrence, Kansas.** C. T. HOUGH. 55–57. Experiences with seasonal variations in efficiency of operation of plant indicate: (1) that ample retention period should be considered in the design of settling basins for the application of chlorine and also for softening treatment for water at low temperatures; (2) that better results will be obtained by double chlorination or by the application of chlorine prior to the application of

lime; and (3) that the effectiveness of chlorine depends upon the contact period as well as the pH value of the water at low temperatures. **Physiological Effects of Minerals Naturally Occurring in Water.** SELMA GOTTLIEB. 58–62. Discussion of the effects upon health of dissolved minerals ordinarily present in water supplies indicates that divergent opinions are held by authorities. **Sources and Significance of Bacteria Found in Water.** CASSANDRA RITTER. 63–67. The organisms commonly found in water, their characteristics, resistance to destruction, and their effects are discussed. **The Importance of Adequate Financial Records for a Water Plant Operator.** W. C. HALL. 68–69. A plea for adequate records, made on the premise that a "definite and accurate record of the total cost of plant operation and of quantity of water pumped is very essential and a most important factor in establishing a rate making basis and in determining the financial showing made by the water department each year." **Sanitary Defects of Ground Water Supplies.** R. E. LAWRENCE. 70–75. Author's conclusion: "statistics indicate that practically all ground-water supplies are subject to regular, intermittent, or occasional contamination, resulting from definite sanitary defects. Such contamination may be of a serious nature, and may result in a great amount of sickness and loss of life. The occasional contamination present in many supplies of excellent sanitary aspects is usually the result of accident. This accidental contamination could in most cases be eliminated entirely by the exercise of more care on the part of the operating personnel. The definite sanitary defects causing intermittent contamination of otherwise safe supplies can in many cases be corrected, and these supplies placed on the safe list. Where the sanitary defects are such that difficulty is experienced in adequately safeguarding a supply, sterilization is required. Extreme cases of polluted ground-water require complete treatment as with surface supplies. It is the purpose of public health authorities and waterworks men to locate and eliminate the sanitary defects causing contamination of ground water supplies. The failure to permanently eliminate all contamination by the correction of physical defects requires in every case sterilization." **Some Hydraulic Data of Interest to Water Works Men.** J. O. JONES. 76–81. A critical analysis of specification, bases of tests, and accuracy of operations of water meters. **Kansas Water Ratings,** 82–85. Ratings for cities, villages by Kansas State Board of Health. **Unusual Uses of Chlorine in a Water Department.** R. W. KEHR. 86–89. The use of chlorine for sterilizing new mains, new wells, and other plant elements subject to contamination is discussed. The remaining pages, 90–125, are devoted principally to matters other than water supply and purification.—*Robert L. McNamee.*

Atlantic Coastal Plain. Surface Water Supply of the United States, 1926; Pt. 1, North Atlantic Slope Drainage Basins. NATHAN C. GROVER, chief hydraulic engineer, H. B. KINNISON, A. W. HARRINGTON, O. W. HARTWELL, A. H. HORTON, and J. J. DIRZULAITIS, district engineers. 1930. U. S. Geol. Survey. Water Supply Paper 621.—*Arthur P. Miller.*

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